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# New developments in field and laboratory testing of soils

## Nouveaux développements des essais in-situ et de laboratoire

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SYNOPSIS A reliable assessment of the stress-strain-time and strength characteristics of natural soil deposits by means of experimental techniques represents the most challenging task of any rational geotechnical analysis and design. Within this context the writers have selected a few relevant topics for detailed discussion. In Chapter 2, laboratory testing, they include: stress history, yielding, normalization, anisotropy, influence of the intermediate principal stress, and for cohesive soils, time effect in oedometer testing. In Chapter 3 the discussion focuses on recent advances in the use of in situ techniques in: soil profiling and identification, assessment of in situ horizontal stress, stress history, deformation, flow and consolidation characteristics of soil deposits. Finally Chapter 4 is devoted to some general aspects of the use of data acquisition systems and microcomputers in geotechnical experimentation.

## 1. INTRODUCTION

### 1.1. OBJECT AND SCOPE

To judge from this title, the present theme lecture has been assigned a very wide spectrum of topics which are best included under the heading of Experimental Soil Engineering. The aim of this branch of Geotechnical Engineering is to obtain -- through experiments performed both in the field and the laboratory -- the following basic design information:

1. Detailed and representative soil profiles, including the description of ground water conditions and soil index properties.
2. Assessment of the initial state of stress existing in the ground and the stress history of the soil prior to any construction activity.
3. Determination of the stress-strain-time and strength characteristics of the encountered soil layers.

The first two points include the initial physical and geometrical conditions of the foundation soils, and it is convenient to group them under a common heading called Initial State Variables (ISV). All the above design information may be obtained by means of a properly planned and executed exploration programme, the scope of which should be related to the complexity of the local geology and the structure to be built.
Successful exploration programmes are generally run in stages. The later and most conclusive stages are linked to the soil model which one intends to use in the design analyses through an appropriate determination of stress-strain-time-strength characteristics.

### 1.2. EXPERIMENTAL SOIL MECHANICS

The methods of ESE* used in geotechnical exploration may broadly be grouped as follows:

1. Laboratory testing, including physical models [Ladd et al. (1977), Saada and Townsend (1981), Schofield (1980)].
2. In Situ testing [Ladd et al. (1977), Mitchell et al. (1978), Mori (1981), Wroth (1984)].
3. Monitoring the field behaviour of full-scale prototypes and/or existing structures by means of Geotechnical Instrumentation which, when properly back-analyzed, enables in many cases the assessment of the pertinent stress-strain-time and strength characteristics of the soil [Di Biagio (1975), Dunnicliff (1982), Hanna (1985)].

Obviously, the number of investigation tools and apparatus belonging to each of the above three groups is almost infinite; a complete review of them is hardly feasible and will not be attempted in the present paper. The intention of the writers is rather to examine in detail a few topics selected on the basis of the following considerations:

1. Priority is given to topics in which the need for research has recently been recognized by the geotechnical community [RNESE (1983), Site Characterization and Exploration (1978)].
2. Recently developed innovative exploration and interpretation methods receive due consideration.
3. Finally, the discussion should be within the context of current research activities at the universities to which the writers belong.

* Experimental Soil Engineering

Discussion of these topics will be made after a brief examination of the following aspects:
- levels of soil behaviour investigations,
- categories of soil models in relation to experiments, and
- laboratory tests versus in situ tests and field measurements.

In the writer's opinion, these aspects are closely interrelated and a proper vision of them is necessary for any soil mechanics experiment to be scientifically valid.

## 1.3. LEVELS OF SOIL BEHAVIOUR INVESTIGATIONS

Generally speaking, the behaviour of soils can be studied at three different levels: micro, macro and mega [Dafalias and Lade in RNESE (1983)], where:

1. The micro-level is the level at which the interaction between single soil particles and clusters of particles are investigated and laws are established which relate the evolution of soil structure and micro-fabric to changes of the imposed stresses and strains in the soil mass [see for example Oda (1972), Cundall et al. (1982), Nemat Nasser (1982), etc.].
2. The macro-level is the level at which quantities such as stress and strain are usually defined and measured by so-called single element laboratory tests like triaxial, plane strain or simple shear tests. At this level, one also formulates constitutive equations for the soil [see for example Roscoe and Burland (1968), Schofield and Wroth (1968), Lade (1977), Prevost (1980), Dafalias and Herrmann (1980) and many others].
3. The mega-level refers to an entire geotechnical structure, including the surrounding soil mass, the behaviour of which may be analyzed by solving the boundary value problem using macro-constitutive equations. A typical example of a soil mass mega-element study is the back-analysis of a case history.

The classification of in situ tests within the scope of these three levels is not straightforward. As far as methods for in situ test interpretation are concerned, they should be assigned to the mega-level since they represent boundary value problems. However, the question of the volume of the soil to be investigated falls between the macro and the mega levels.

## 1.4. ANALYTICAL PROCEDURES AND SOIL MODELS

The analysis of geotechnical problems usually requires the choice of an appropriate computational method which is strictly linked to the adopted soil constitutive model. The model, in turn, is related to a specific set of soil parameters to be determined through an appropriate exploration programme. This means that there must be a direct connection between the method of analysis which the designer intends to use and the type and extent of the proposed soil exploration.
Following the RNESE (1983), the presently available analytical methods may be grouped as shown in Table I.

## 1.5. LABORATORY VERSUS FIELD MEASUREMENTS

Conventional exploration programmes make use of both in situ and laboratory measurements for the determination of soil properties, the latter being based on the testing of so-called undisturbed samples obtained from borings and exploration pits. It is quite common to find that geotechnical engineers declare themselves in favour of one of these basic exploration methods, without considering that the suitability of one approach or the other is strictly linked to the aims of the exploration and the method of analysis which is intended to be used in the design. In fact, both methods of soil investigation have their inherent merits and disadvantages which are briefly summarized here:

LABORATORY TESTS

1. <u>Advantages:</u>

- Well-defined boundary conditions.
- Strictly controlled drainage conditions.
- Preselected and well-defined stress paths are followed during the tests.
- In principle, uniform strain fields (this assumption is acceptable for small strain levels only and soils which do not exhibit strain-softening behaviour) are imposed on the specimens, which allows the application of continuum mechanics theories to the interpretation of test results.
- Soil nature and physical features are positively identified.

2. <u>Limitations:</u>

- In cohesive soils, the effects of unavoidable sample disturbance in even so-called "high quality" undisturbed samples are sometimes difficult to assess.

TABLE I

Categories of Analytical Methods for Soil Studies

| Category | Main Features of Models | Determination of Soil Parameters |
|---|---|---|
| I | Very advanced models using nonlinear elastic plastic time dependent laws which possibly incorporate anisotropic behaviour | Only from sophisticated laboratory tests, with the exception of the ISV which must be obtained from in situ tests |
| II | Advanced models using constitutive incremental elasto-plastic laws and nonlinear elastic relationships | Laboratory tests which are only a little more sophisticated than conventional tests. In situ tests are also appropriate, particularly as far as the ISV are concerned |
| III | Simple continuum, such as isotropic elastic continuum, including layering and empirical models | Conventional laboratory and in situ tests |

- In cohesionless soils, undisturbed sampling is still an unsolved problem in everyday practice.
- The small volume of laboratory specimens [Rowe (1972)] cannot incorporate the frequently present macrofabric and inhomogeneities of natural soil deposits. This leads to doubts as to what extent the field behaviour of a large soil mass can be successfully modelled by small scale laboratory tests.
- The factors causing the formation of shear planes during the testing of laboratory specimens are still very poorly understood. Shear planes are frequently associated with such phenomena as induced shear stresses, soil volume changes, nonuniformity of laboratory specimens and consequent nonuniform strain distributions, boundary and kinematic constraints, and stress concentrations imposed by the laboratory apparatus [RNESE (1983)]. It must be emphasized that once a shear plane has developed in a laboratory specimen, deformations are concentrated along this plane and displacements and stresses measured at the specimen boundaries are consequently no longer a function of the stress-strain behaviour of the tested material.
- In principle, the discontinuous nature of information obtained from laboratory tests may lead to erroneous modelling of the behaviour of a large soil mass.
- In general terms, soil explorations based on the laboratory testing of soil samples from borings are likely to be more expensive and time-consuming than explorations which make use of in-situ testing techniques.

A special kind of laboratory test is represented by centrifuge models. Well-established scaling laws make it possible to obtain reasonable similirities between test model and prototype structure [Rowe (1975), Schofield (1980, 1983), Scott (1983)]. Centrifuge models are particularly useful in studying the behaviour of complex geotechnical structures where the mode of deformation and near-failure kinematics are unknown. Through appropriate parametric studies, centrifuge testing permits the verification of existing computation methods and the improvement of numerical models where geometry and loading conditions are complex.
No further attention will be devoted to this important and extremely promising physical modelling technique, since the present conference provides for a special discussion session on centrifuge testing.

## IN SITU TESTS

1. Advantages:

- A larger volume of soil is tested than is usually done in most laboratory tests; hence in situ tests should in principle reflect more accurately the influence of the macrofabric on the measured soil characteristics.
- Many devices currently in use produce a continuous record of the soil profile which allows the soil macrofabric and layer boundaries to be determined.
- In situ tests can be carried out in soil deposits in which undisturbed sampling is still impossible or unreliable. Examples include cohesionless soils, soils with highly-developed macrofabrics, intensively layered and/or heterogeneous soils, and highly fissured clays.
- The soils are tested in their natural environment which may not be preserved in laboratory tests. For example, the most successful attempts to measure the existing initial total in situ horizontal stress are the recent developments in situ techniques, e.g., self-boring pressure meter (SBP), flat dilatometer (DMT), Iowa stepped blade (ISB), spade-like total stress cells (TSC).
- In general terms, soil exploration by means of in situ techniques is more economical and less time consuming than investigations based on laboratory tests.

2. Limitations:

- Boundary conditions in terms of stresses and/or strains are, with the possible exception of the self-boring pressuremeter (SBP), poorly defined, and a rational interpretation of in situ tests is very difficult.
- Drainage conditions during the tests are generally unknown and make it uncertain if the derived soil characteristics reflect undrained, drained or partially drained behaviour. In this respect, quasistatic cone penetration tests with pore pressure measurements (CPTU) and SBP tests (also with pore pressure measurements), when properly programmed, help to minimize the problem.
- The degree of disturbance caused by advancing the device in the ground and its influence on the test results is generally (with the possible exception of the SBP) large but of unknown magnitude.
- Modes of deformation and failure imposed on the surrounding soil are generally different from those of civil engineering structures; furthermore, they are frequently not well established, as for example in the field vane (FV) test.
- The strain fields are nonuniform and strain rates are higher than those applied in laboratory tests or those which are anticipated in the foundation on structure.
- With the exception of the Standard Penetration Test (SPT), the nature of the tested soil is not directly identified by in situ tests.

The limitations of in situ tests lead to a situation in which almost all present interpretation techniques are empirical, with the exception of the methods for the SBP test.

## BACK-ANALYSES OF FULL SCALE PROTOTYPES

In the last two decades, the rapid growth of geotechnical instrumentation and its widespread application has made available a number of well-documented case records of the field behaviour before and at failure of a wide variety of geotechnical construction and full scale prototypes. The importance of the publication of well-documented case records led the University of Missouri-Rolla in 1984 to organize a special conference in St. Louis on Case Histories In Geotechnical Engineering. An analysis of the more than 150 papers presented to that conference confirms that measured field performance data are used in the two ways postulated by Lambe (1973):

1. As controlled boundary value problems, to verify a specific constitutive soil model and the related computation procedure. In this case, all input parameters are determined from appropriate laboratory tests which are supposed to represent the stress-strain-time behaviour and strength characteristics of the soils underneath the structure being monitored.

2. As a set of experimental data such as pore pressures, vertical and horizontal displacements,

vertical strains, etc. which make it possible, within the frame of the adopted model of soil behaviour, to assess the relevant soil parameters.

Of these two approaches, only the second one is pertinent to the scope of the present theme lecture.
The writers have no doubt that since the publication of the classic works by Terzaghi (1950), Skempton and Brown (1961), Skempton and La Rochelle (1965), Peck (1969), Bjerrum (1972), Lambe (1973), this is the procedure which, in principle, leads to the most reliable assessment of geotechnical design parameters (if compared only to what may be obtained from the results of laboratory and in situ tests).
However, as often happens in practice, the self-evident may not be sufficient and reliable soil parameters may not be guaranteed by a back-analyses of field records alone. In fact, this type of procedure often requires many simplifying assumptions which present the designer with frequent pitfalls, as has been illustrated by Leroueil and Tavenas (1981). Generally involved is one or more of the following:

1. Soil constitutive model and related computational procedure.
2. Drainage conditions and boundaries.
3. Initial state variables of the considered problem, including soil stratigraphy, ground water conditions, initial stress state and stress history of the soil deposit, etc.
4. Auxiliary soil characteristics which frequently must be introduced in a back-analysis. This means that the examined problem is controlled by a larger number of parameters than the analysis itself can yield.

The sensitivity of back-analyzed soil characteristics is not always evaluated with respect to a given set of assumptions. In addition, the designer must fully understand that a substantial difference exists between soil parameters derived from a back-analysis of field data which reflects the performance of a single point (e.g. stress, strain, pore pressure, etc.) and those obtained from the analysis of average foundation conditions (e.g. load-displacement curves, etc.). It must be borne in mind that only the former type of back-analyses of existing case records enables us to obtain soil parameters which may be directly compared to the ones inferred from laboratory and in situ tests.
The above brief summary of the limitations and advantages of the three basic methods of ESE clearly indicates that their capability for a positive and comprehensive identification of the engineering properties of foundation soils is complementary. To support this statement, the current capabilities of the three methods are described in Tables II through IV.

## 1.6. SUMMARY AND SELECTED TOPICS

1. Only laboratory tests make it possible to investigate such complex features of soil behaviour as initial and evolving anisotropy of both strength and deformability, the effects of stress and/or strain reversal, time effects, the influence of intermediate principal stress, etc. Therefore, only those tests which examine the soil at a macrolevel allow the determination of parameters which characterize realistic constitutive soil models.
2. The role of in situ tests in experimental soil mechanics consists mainly in the assessment of:
   - Initial state variables and particularly detailed soil profiles, but also the in situ initial lateral stress, $\sigma_{ho}$; in this respect, the limitations of laboratory tests are particularly pronounced.
   - Drained and undrained soil stiffness, especially at small and moderate strain levels. The determination of this parameter in the laboratory is an almost impossible task for cohesionless soils and often of questionable reliability for cohesive materials due to the well known sensitivity of these parameters to even small sample disturbance.
   - Flow and consolidation properties, especially in cohesionless deposits and cohesive soils with a well-developed macro-fabric. The unreliability of laboratory tests in these conditions is widely recognized.
3. The monitoring of the field behaviour of construction and full-scale prototypes makes it possible, if properly planned, controlled and analyzed to verify constitutive soil models and related analytical and numerical methods, and to assess, at a megalevel, relevant soil characteristics like strength and deformability, by by-passing most of the limitations inherent in laboratory and in situ tests.
4. In all circumstances, the three methods of ESE mentioned above are complementary, and the relative importance of each is strictly linked to local geology, the type of construction, and the menthod of analysis which one intends to adopt in a specific design.

In relation to the above and taking account of the priority research needs in ESE as identified during the USA NSF Workshop on Research Needs in Experimental Soil Engineering (1983), the following topics have been selected for coverage herein:

### Laboratory Testing

1. Preconsolidation pressure, yielding and normalization.
2. Assessment of sample disturbance and procedures to minimize its effects.
3. Strength deformation characteristics under generalized states of stress.
4. Time effects.

### In Situ Testing

1. The assessment of the initial state variables, with particular reference to the initial in situ lateral stress and soil profiling.
2. The evaluation of soil stiffness with particular reference to deformability parameters under drained conditions.
3. A critical review of in situ methods used for the evaluation of flow and consolidation characteristics.

### Instrumentation

Due to the already wide spectrum of topics of this theme lecture and due to the obvious space limitations, the writers have decided not to cover in detail topics related to geotechnical instrumentation, with the exception of data acquisition systems and micromputer applications in geotechni-

TABLE II

Present Laboratory Capabilities for Soil Modelling

| Soil Behaviour-Parameter | Equipment and/or Procedures | Comments - Remarks |
|---|---|---|
| 1. In situ $K_o$ | 1. $K_o$ vs. OCR TX and OED tests, latter with allowance for lateral stress measurements | 1.1.a. Restricted to cohesive soils<br>1.1.b. Applicable only to mechanical overconsolidation assuming normalized behaviour |
| 2. In situ vertical effective yield stress $\sigma'_p$ | 2. Mainly OED tests | 2.1.a. Requires good quality cohesive undisturbed samples<br>2.1.b. Very difficult to distinguish among different overconsolidation mechanisms |
| 3. Influence of intermediate principal stress $\sigma_2$ on strength and stiffness | 3.1. True TX tests with rigid, flexible and mixed boundaries<br>3.2. TSHC with different internal $p_i$ and external $p_e$ pressures | 3.1. Quite extensive data for sands yielding conflicting results for high $b=(\sigma_2-\sigma_3)/(\sigma_1-\sigma_3)$ values<br>3.2. Rather limited data for clays |
| 4. Initial anisotropy with respect to peak strength and stiffness | 4.1. TX or PS tests on inclined specimens<br>4.2. True TX tests<br>4.3. TSHC<br>4.4. DSC | 4.1. Questionable validity<br>4.2. Limited to $\delta=90°$ rotation of principal stresses at failure<br>4.3.a. "Proven" for CU and CD tests on sands and CU tests on clays<br>4.3.b. Can explore full range of $0° \leq \delta \leq 90°$ (in theory)<br>4.4.a. Can explore full range of $\delta$ with plane strain conditions<br>4.4.b. "Proven" for sands and OC clays; further development needed for NC clays<br>4.4.c. Limited to low shear stress levels |
| 5. Evolving anisotropy and effect of large stress reversal | 5.1. True TX tests<br>5.2. TSHC tests<br>5.3. DSC | 5.1. Limited to $\delta=90°$ rotation of principal stresses at failure<br>5.2. Excellent theoretical potential<br>5.3.a. Fairly versatile within the limits of plane strain conditions<br>5.3.b. Extensive data for induced anisotropy in sands available |
| 6. Time effects (creep and strain rate) | 6.1. OED and almost all TX, PS, DSS, true TX and TSHC tests can investigate time effects | 6.1.a. Experimental problems due to membrane leakage, friction on the specimens boundaries and temperature fluctuations<br>6.1.b. Lack of experimental data for varying stress conditions |
| 7. Influence of confining stress level on peak strength and stiffness | 7.1. Few practical limitations with conventional laboratory equipment (TX, PS, DSS, TRC, TSHC). Restrictions arise for devices like DSC | 7.1.a. Important for dense sands, stiff clays and cemented soils<br>7.1.b. There is a limitation in the DSC: the applied shear stress cannot exceed 50 kPa. |
| 8. Small-strain ($\gamma<10^{-3}$%) shear modulus and internal soil damping | 8.1. TRC, TSHC tests | 8.1.a. "Proven" for clays and laboratory reconstituted sands; limited data available for undisturbed sand<br>8.1.b. In clays the results are very sensitive to even small sample disturbance<br>8.1.c. Majority of the available data obtained on isotropically consolidated specimens<br>8.1.d. Very little known about influence of initial anisotropy and $\sigma_2$ on measured G |

SYMBOLS:

$\sigma'_p$ = Maximum past vertical effective stress
OED = Oedometer apparatus
DSS = Direct simple shear apparatus
TRC = Torsional resonant column apparatus
PS = Plane strain apparatus
CD = Consolidated drained
OCR = Overconsolidation ratio
TX = Triaxial apparatus
TSHC = Torsional shear hollow cylinder apparatus
DSC = Directional shear cell
CU = Consolidated undrained
$K_o$ = Coefficient of earth pressure at rest

cal engineering (Chapter 4 ). The writers believe that most readers will realize the relevance of proper design, selection, calibration, installation and interpretation of geotechnical instrumentation.

## 2. LABORATORY TESTING

### 2.1. INTRODUCTION

In this section the writers' focus is on equipment and procedures for testing cohesive soils because most practical problems involving clays usually require at least some design parameters from laboratory testing.

In planning any laboratory program, it should be recognized that the in situ undrained and drained behaviour of a clay is largely controlled by the magnitude of the applied stresses in relation to the "yield envelope" for the soil. Section 2.2. discusses the practical aspects of this concept, especially regarding the influence of stress history and clay sensitivity. Section 2.3. deals with procedures for assessing and minimizing sample disturbance in cohesive soils. The topic of the measurement and influence of stress-strain-strength anisotropy is treated in Section 2.4. The final section discusses "time effects" during consolidation.

TABLE III
Present In Situ Capabilities for Soil Modelling

| Soil Behaviour-Parameter | Equipment and/or Procedures | Comments - Remarks |
|---|---|---|
| 1. Soil profiling and identification | 1.1. CPTU | 1.1.a. Simultaneous measurement of $q_c$ and $u_{max}$ during penetration has great potential for soil profiling and identification |
| | | 1.1.b. Essentially rigid and extremely well deaired system with a very quick response for reliable $u_{max}$ measurements |
| | | 1.1.c. Correction of $q_c$ and $f_s$ for unequal end areas effects |
| | 1.2. CPT | 1.2.a. Good for soil profiling but less sensitive to strata changes in comparison to CPTU |
| | | 1.2.b. Friction ratio $f_s/q_c$ a poor soil type identifier in especially sensitive clays. |
| | | 1.2.c. Potential may be increased by improving resolution of $q_c$ measurements and more reliable and repeatable $f_s$ measurements |
| | 1.3. DMT | 1.3.a. $I_D$ a sensitive soil identifier but, since performed discontinuosly, generally every 20 cm, less sensitive to strata changes |
| | 1.4. Acoustic cone | 1.4.a. Mainly for soil profiling and identification; needs further field and laboratory validation |
| | 1.5. Electric conductivity probe | 1.5.a. Measures nondimensional electrical "formation factor" which reflects sand structure, hence its anisotropy, particle shape, void ratio, and cementation; may be relevant for liquefaction studies |
| | | 1.5.b. Needs further validation, especially in the field. |
| 2. In situ $\sigma_{ho}$ hence $K_o$ | 2.1. SBP (measures $\sigma_{ho}$) | 2.1.a. "Proven" to be successful in soft clays; less experience in stiff clays; poor experience in sands |
| | | 2.1.b. Greatest potential among in situ methods but still some problems with equipment compliance and probe insertion procedures |
| | 2.2. DMT (assesses $K_o$) | 2.2.a. Based on empirical correlations; promising, but requires further research to assess reliability |
| | 2.3. ISB (assesses $\sigma_{ho}$) | 2.3.a. New device; requires further intensive laboratory and in situ validation |
| | 2.4. Spade-like TSC (measures $\sigma_{ho}$) | 2.4.a. Limited positive experience only in soft to stiff clays; successful use in other soils unlikely |
| | | 2.4.b. In stiff clay overestimates $\sigma_{ho}$; requires correction for bedding error |
| | | 2.4.c. Vertical installation essential |
| | 2.5. Hydraulic fracturing, (assesses $\sigma'_{ho}$) | 2.5.a. Applicable only to cohesive soils having $K_o < 1$ |
| | | 2.5.b. Interpretation uncertain. |
| 3. In situ vertical effective yield stress $\sigma'_p$ | 3.1. PL and SPL tests | 3.1.a. Limited experience for assessment of $\sigma'_p$ |
| | | 3.1.b. Possible applications limited to relatively homogeneous cohesionless deposits at shallow depths in which tests are performed under fully drained conditions. |
| | | 3.1.c. For SPL the influence of plate shape and disturbance due to its installation on the load-settlement relationship not well understood |
| | 3.2. CPTU | 3.2.a. $u_{max}-u_o/q_c-\sigma'_{vo}$ may be correlated to OCR; in homogeneous cohesive deposits, reflects OCR changes |
| | | 3.2.b. Possible applications limited to cohesive deposits; further laboratory and field validation needed |
| 4. Deformability characteristics | 4.1. PL and SPL | 4.1.a. Application limited to shallow depths |
| | | 4.1.b. "Proven" in cohesionless deposits in which can determine average drained Young stiffness E', within the depth of influence of the plate |
| | | 4.1.c. In cohesive soils, despite uncertainty about drainage conditions, it is assumed to yield average undrained Young stiffness $E_u$ |
| | | 4.1.d. Since E is obtained from load-displacement measurements, an a priori assumption regarding soil constitutive model is necessary |
| | | 4.1.e. Very difficult to refer the E obtained from PL and SBL tests to the behaviour of a soil macro-element, hence to strain or stress levels |
| | 4.2. SBP tests | 4.2.a. Great potential for direct measurement of shear modulus $G_h$ in horizontal direction |
| | | 4.2.b. $G_h$ describing the "elastic" soil behaviour can be assessed from small unloading-reloading cycles whose role is to minimize the soil disturbance due to probe insertion |
| | 4.3. DMT | 4.3.a. Through empirical correlations yields values of tangent constrained modulus in sands and clays |
| | | 4.3.b. Presently available correlations have been obtained mainly for predominantly quartz sands and marine and alluvial clays; further laboratory and field validation in a wider range of soils needed. |
| | 4.4. CPT | 4.4.a. Empirical correlations between $q_c$ and E of questionable reliability and not generally valid except for NC sand |
| | | 4.4.b. In any case applicable only to predominantly quartz clean uncemented sands in which penetration occurs under fully drained conditions |
| | 4.5. Shear wave velocity measurements | 4.5.a. "Proven" potential to evaluate small strain ($\gamma<10^{-3}\%$) G in horizontally layered soil deposits |
| | | 4.5.b. The value of G is calculated after assumption are made concerning the constitutive soil model, the travel path and the soil homogeneity |

TABLE III CONT.

| Soil Behaviour-Parameter | Equipment and/or Procedures | Comments - Remarks |
|---|---|---|
| 5. Flow and consolidation | 5.1. Borehole | 5.1. Outflow tests at constant head preferred; interpretation above water level extremely complex |
| | 5.2. Large scale pumping | 5.2. Very reliable but also very expensive test; accurate well installation and drawdown measurements with piezometers are required |
| | 5.3. Piezometers | 5.3. Constant head tests with $\Delta u$ small to avoid fracturing are preferred; parameters from outflow tests relevant to OC conditions; inflow tests appropriate for NC conditions |
| | 5.4. Self-boring permeameter | 5.4. Careful installation required; only outflow tests available, hence the derived parameters are relevant to OC conditions |
| | 5.5. Holding test (Camkometer) | 5.5. Careful installation required; difficult interpretation due to non-monotonic changes of effective stress |
| | 5.6. Piezocone or piezometer probe | 5.6. Very economical and great repeatibility; great care required when performing test and interpretating field measurements |
| | 5.7. Back-analysis of full-scale structures | 5.7. Uncertainties related to initial excess pore pressure or to final consolidation settlement; methods based on consolidation rate need careful analysis of experimental data |

SYMBOLS:

CPTU = Quasi-static cone penetration test with pore pressure measurements

CPT = Quasi-static cone penetration test

DMT = Marchetti's flat dylatometer

ISB = Iowa stepped blade

TSC = Total stress cell

SBP = Self boring pressuremeter

PL = Plate loading

SPL = Screw plate loading

$\sigma_{ho}$ and $\sigma'_{ho}$ = Respectively, total and effective in situ horizontal stresses

$\sigma'_{vo}$ = Effective overburden stress

TABLE IV

Possible Current Contributions of Field Instrumentation to Advanced Soil Modelling

| Soil Behaviour-Parameter | Equipment and/or Procedures | Comments - Remarks |
|---|---|---|
| 1. Operational strength | 1.1. Lateral soil deformation measured below embankments and in slopes (vertical inclinometer tubes) which suffered a bearing capacity type failure | 1.1.a. Detection of location of failure surface; hence shear strength may be inferred<br>1.1.b. Probably the most reliable way to assess the average operational shear strength of slopes and embankments on clays which have failed<br>1.1.c. A priori assumptions concerning failure kinematics; a constitutive model for the soil is needed |
| 2. Deformability characteristics | 2.1. Measurement of vertical soil displacement, $u_z$, at single points, within the soil mass under loaded area | 2.1.a. With at least two values of $u_z$ measured at two elevations in the same layer, the vertical strain and, consequently, the average E of the soil between the two points may be assessed<br>2.1.b. The distribution of the induced total stresses must be assessed, since an a priori assumption regarding the constitutive behaviour of the soil is necessary<br>2.1.c. In order to refer the assessed E to known drainage conditions, the excess pore pressure in the considered layer should be monitored |
| | 2.2. Measurement of surface displacement under loaded area | 2.2.a. Poor precision of derived parameters, hence of limited research value<br>2.2.b. Yields an average E within the depth of influence of the loaded area<br>2.2.c. In order to properly "qualify" the assessed stiffness, the excess pore pressures and lateral soil displacements below the loaded area should be known<br>2.2.d. A priori assumption regarding the soil constitutive model is necessary<br>2.2.e. It is almost impossible to link the obtained E to the stress or strain levels of the representative macro-element of soil |
| 3. Consolidation characteristics | 3.1. Decay of excess pore pressure with time observed in piezometers | 3.1.a. The coefficient consolidation may be estimated for vertical flow ($c_{vv}$) in the absence of vertical drains or for horizontal flow ($c_{vh}$) in the presence of vertical drains<br>3.1.b. Despite the necessity to assume a priori a soil constitutive model, this is one of the most reliable ways to assess $c_{vv}$ and $c_{vh}$ |
| | 3.2. Measurement of $u_z$ at single points within soil mass under loaded area | 3.2.a. As mentioned under point 2.1.a., the variation of vertical strains in the monitored layer may be estimated and, hence, $c_{vv}$ or $c_{vh}$ may be inferred<br>3.2.b. For a proper interpretation of the strain evolution with time, the excess pore pressure and lateral soil deformations beneath the loaded area should be monitored<br>3.2.c. An a priori assumption concerning the soil constitutive behaviour is unavoidable |

## 2.2. PRECONSOLIDATION PRESSURE, YIELDING AND NORMALIZATION

### 2.2.1. Introduction

Casagrande (1936) defined the preconsolidation pressure, $\sigma'_p$, as the "largest overburden beneath which the soil had once been consolidated". He also presented an empirical technique to estimate the value of $\sigma'_p$ from the shape of the void ratio versus logarithm vertical consolidation stress curve obtained from incremental one-dimensional consolidation (oedometer) tests performed on undisturbed samples. During the 1960's it became evident that the Casagrande $\sigma'_p$ profile for some clay deposits was higher than the "maximum past pressure" (maximum vertical consolidation stress) believed to have existed during geological history. This discrepancy was attributed to various "soil structure" phenomenon, such as long term secondary compression, aging and cementation effects [e.g. Leonards and Altschaeffl (1964), Bjerrum (1967)]. Although special terms were suggested to designate these causes, it is difficult to establish for most clay deposits exactly when they would apply. Hence, while "preconsolidation pressure" may often be a misleading term in the physical sense, it remains the accepted term to denote the "break" in the oedometer curve. As such, it really represents a yield stress that separates small strain "elastic" behaviour from large strains accompanied by plastic (irrecoverable) deformation during one-dimensional compression. The writers will use $\sigma'_p$ = preconsolidation pressure in this context irrespective of the physical cause.

Although the profession can argue over terminology, we should all accept the view that an accurate determination of the $\sigma'_p$ profile is generally the single most important step in predicting long term consolidation settlements. The writers believe that it is equally important for most short term stability problems. Hence we will first review procedures for measurement of $\sigma'_p$ and the different mechanisms responsible for developing the in situ $\sigma'_p$ profile. This is followed by an overview of the concept of a yield envelope, its practical implications and how such envelopes can be affected by the $\sigma'_p$ mechanism. Finally, we will emphasize the usefulness of normalizing undrained stress-strain-strength parameters with respect to $\sigma'_p$. In doing this, the term stress history is used to denote the in situ effective overburden stress, $\sigma'_{vo}$, and preconsolidation pressure, $\sigma'_p$, profiles. This term has the same problems as already discussed for $\sigma'_p$, as does the overconsolidation ratio, OCR = $\sigma'_p/\sigma'_{vo}$.

### 2.2.2. Preconsolidation Pressure

Major variables that can influence the measured $\sigma'_p$ obtained from laboratory tests run on undisturbed samples can be divided into four categories:

1. Sample disturbance which will almost always reduce the measured value of $\sigma'_p$.
2. The test equipment and procedures used to obtain the one-dimensional compression curve and related time effects.
3. The interpretative technique used to estimate the value of $\sigma'_p$.
4. Environmental factors such as pore fluid composition and temperature (a 10°C increase in temperature can cause a significant reduction in $\sigma'_p$ for some clays; see Ladd et al. 1977, page 442).

Regarding 2., the incremental oedometer test using a load increment ratio $\Delta P/P$ of unity is most common, except that $\Delta P/P$ is sometimes reduced to about 0.5 to obtain a better defined curve in the vicinity of $\sigma'_p$ (eastern Canada uses 0.5 throughout for its sensitive clays). Each stress increment is typically maintained for one day, and most practitioners use the 24 h compression curve to estimate $\sigma'_p$. Since the end of primary usually occurs within less than 1 h, the virgin portion of the 24 h curve is displaced downward by one or more cycles of secondary compression. Others such as Ladd (1973), Mesri and Choi (1985) and ASTM (D2435-80) recommend using end of primary, EOP, curves as discussed in Section 2.5. In any case, conventional oedometer tests require considerable time and yield discontinuous compression data, which led to the development of the controlled gradient (CG) test [Lowe et al. (1969)] and the constant rate of strain (CRS) test [Smith and Wahls (1969), Wissa et al. (1971), Sällfors (1975), ASTM (D4186-82)]. Both tests require measurement or control of boundary pore pressures and give "during primary" compression curves since excess pore pressures exist throughout testing. The CG test also involves a change in strain rate near $\sigma'_p$, which can significantly affect the compression curve, due to changes in compressibility and permeability.

Based on data summarized in Section 2.5., it will be concluded that:

1. The end of primary (EOP) compression curve for incremental loading is independent of sample (drainage) thickness (at least for drainage heights equal to or greater than used in conventional laboratory oedometer tests) [Mesri and Choi (1985)].
2. One day compression curves from standard incremental oedometer tests yield values of $\sigma'_p$ generally 10 ± 10% lower than EOP curves due to increased strains resulting from secondary compression.
3. Controlled gradient (CG) and constant rate of strain (CRS) consolidation tests run at rates significantly higher than that corresponding to the end of primary compression in conventional oedometer tests can result in values of $\sigma'_p$ higher than obtained from oedometer EOP curves [Leroueil et al. (1983)].

#### Interpretative $\sigma'_p$ Techniques

Various interpretative techniques have been proposed to estimate $\sigma'_p$ from a given compression curve, (i.e. the void ratio or strain and $\sigma'_{vc}$ data), the most common still being Casagrande's (1936) empirical e or $\varepsilon$ versus log $\sigma'_{vc}$ construction, Fig.1(a). Note that the location of the minimum radius point can be influenced by the scale, which should be standardized, and involves judgement with gently rounded curves (disturbed samples, very stiff clays, etc.). Schmertmann's (1955) technique is considered advantageous in estimating $\sigma'_p$ for overconsolidated clays wherein sample disturbance is suspected [Peck (1974)].

Reliance on semi-log plots to obtain $\sigma'_p$ has been criticized by Janbu and Senneset (1979) and others who maintain that arithmetic diagrams of stress versus strain (or modulus $1/m_v$, or coefficient of consolidation, etc.) are better indicators of changes in basic behaviour. The writers agree that high quality samples frequently show an essentially linear stress-strain relationship during recompression. However, except for highly structured soils that undergo a dramatic increase in compres

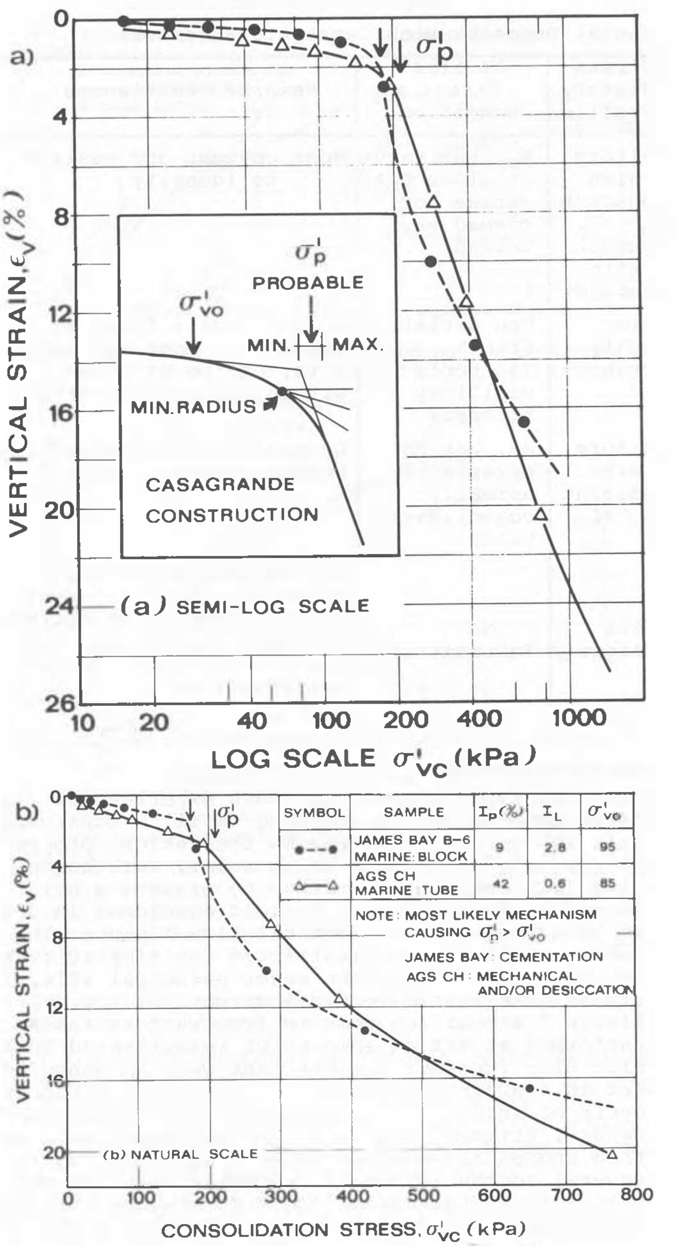



Fig.1: Compression Curves for Clays Having Different Preconsolidation Pressure Mechanisms (a) Semi-Log Scale; (b) Natural Scale.

sibility beyond $\sigma'_p$ (e.g. the James Bay clay in Fig.1(b)), the resultant range in the probable $\sigma'_p$ if often much greater than can be reasonably estimated from the conventional semi-log plot.

## Preconsolidation Pressure Mechanisms

The mechanism(s) responsible for causing the observed preconsolidation pressure of horizontal clay deposits can have several practical implications, as summarized in Table V. Mechanical one-dimensional loading-unloading typically leads to a uniform amount of precompression (constant $\sigma'_p - \sigma'_{vo}$) and $K_o$ conditions, although $K_o$ at a given OCR depends on whether or not $\sigma'_{vo}$ has been increased or decreased to its present value.

Desiccation due to drying, freeze-thaw cycles, etc. will usually produce scattered, often difficult to define values of $\sigma'_p$ and the in situ stresses may deviate from $K_o$ conditions, e.g. isotropic stresses from evaporative drying. The generation of a preconsolidation pressure due to aging, defined herein as long term one-dimensional drained creep (=secondary compression), is certainly well documented in the laboratory [Leonards and Altschaeffl (1964)] and is supported by case histories [Bjerrum (1967)]. It should result in a constant OCR, but whether $K_o$ remains constant during secondary compression is in dispute (see Section 2.5.). Finally, it is now generally accepted that various physio-chemical phenomena can cause an increase in $\sigma'_p$, particularly natural cementation due to carbonates, silica, ion exchange, etc. The resultant $\sigma'_p$ profile is likely to be variable, as illustrated in Fig.2 for a deposit of James Bay marine clay. Although the in situ $K_o$ may remain constant during development of $\sigma'_p$, the yield stress for horizontal loading would presumably increase due to cementation. It should be noted that cementation can be significant in deposits ranging from heavily overconsolidated clay shales [McKowen and Ladd (1982)] to the brittle quick clays of Canada.

## Discussion

We pose two questions: First, what is the correlation between $\sigma'_p$ measured in the laboratory and that observed in the field? Based on five case histories involving the highly structured Canadian Champlain clays, Leroueil et al. 1983a,b) and Morin et al. (1983) conclude that the in situ $\sigma'_p$ for these low to moderate OCR clays (OCR < 4.5) is within about ±10% of that obtained from conventional oedometer one day curves run on 70 mm piston samples. But comprehensive field data on a fairly similar deposit in New England suggest a better correlation is with oedometer end of primary (EOP) $\sigma'_p$ values. However, given the lack of definitive data on a variety of clays, no answer to the question can be made other than the fact that controlled gradient (CG) and constant rate of strain (CRS) tests run at rates significantly faster than the corresponding EOP rate in oedometer tests can seriously overestimate the in situ $\sigma'_p$ (assuming negligible sample disturbance). Nevertheless, the writers concur with Mesri's (1984) view that it is conceptually more correct to use EOP rather than one day curves from oedometer tests, even though the difference is generally less than 10 to 20%.

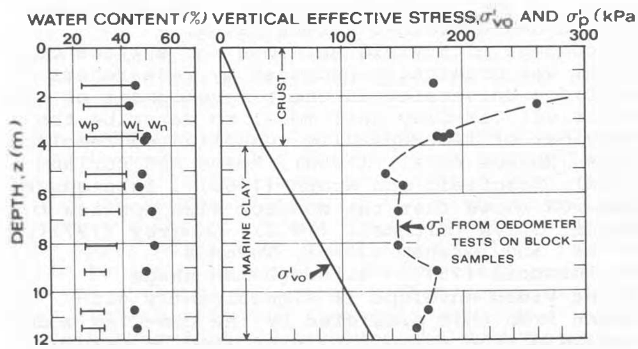



Fig.2: Stress History of James Bay B-6 Marine Clay [Data Replotted from Lefebvre et al. (1983)].

TABLE V

Preconsolidation Pressure Mechanisms (For Horizontal Deposits with Geostatic Stresses)

| Category | Description | Stress History Profile | In situ Stress Condition | Remarks / References |
|---|---|---|---|---|
| A) Mechanical One Dimensional | 1) Changes in total vertical stress (overburden, glaciers, etc.)<br>2) Changes in pore pressure (water table, seepage conditions, etc.) | Uniform with constant $\sigma'_p - \sigma'_{vo}$ (except with seepage) | $K_o$, but value at given OCR varies for reload vs. unload | Most obvious and easiest to identify |
| B) Desiccation | 1) Drying due to evaporation vegetation, etc.<br>2) Drying due to freezing | Often highly erratic | Can deviate from $K_o$, e.g. isotropic capillary stresses | Drying crusts found at surface of most and deposits; can be at depth within deltaic deposits |
| C) Drained Creep (Aging) | 1) Long term secondary compression | Uniform with constant $\sigma'_p / \sigma'_{vo}$ | $K_o$, but not necessarily normally consolidated value | Leonards and Altschaeffl (1964); Bjerrum (1967) |
| D) Physico-Chemical | 1) Natural cementation due to carbonates, silica, etc.<br>2) Other causes of bonding due to ion exchange, thixotropy, "weathering" etc. | Not Uniform | No Information | Poorly understood and often difficult to prove. Very pronounced in eastern Canadian clays, e.g. Sangrey (1972), Bjerrum (1973), Quigley (1980) |

The second question is: how to deal with scattered $\sigma'_p$ data from laboratory tests (assumed to be of one type)? We have no simple answer, but do offer the following advice: First attempt to separate out effects of varying degrees of sample disturbance (Section 2.3); and then evaluate the data in light of the most likely mechanisms responsible for development of the preconsolidation pressure. Mechanisms A) and C) in Table V should usually produce relatively linear $\sigma'_p$ profiles, whereas B) and D) are likely to cause true spatial variability.

2.2.3. Yield Envelopes

As previously discussed, the real physical significance of the preconsolidation pressure is that it represents the yield stress observed during one-dimensional drained loading. It also represents one point on the yield envelope, defined as the locus of stress states that separates small strain, "elastic" behaviour from large strain, plastic (irrecoverable) behaviour for different drained and undrained stress paths.

The concept of a yield envelope (or surface or locus) was originally proposed by researchers at Cambridge University in their development of the (modified) Cam-Clay soil model to describe the behaviour of isotropically consolidated remolded clays [Roscoe et al. (1958), Rosce and Burland (1968), Schofield and Wroth (1968)]. Subsequent research shows that the concept also applies to natural clays [Mitchell (1970), Sangrey (1972), Crooks and Graham (1976), Tavenas and Leroueil (1977)] although the shape of the yield envelope is significantly different from that predicted by the Cam-Clay models because of the anisotropy exhibited by natural clays. New soil models used in advanced finite element analyses now incorporate anisotropic yield surfaces [e.g. Prevost and Höeg (1977), Prevost (1978), Kavvadas and Baligh (1982)].

Our objective is not to discuss experimental criteria used to define yielding (still a controversial subject), nor to review theoretical properties of yield surfaces (flow rules, work hardening laws, etc.), but rather to present a brief overview of the nature of yield envelopes in order to illustrate their practical significance. The presentation will be restricted for simplicity to stress paths wherein the major principal effective stress acts in the vertical direction, i.e. $\sigma'_1=\sigma'_v$.

Figure 3 summarizes results from various tests performed at MIT on samples of resedimented Boston Blue clay ($w_L$=40%; $I_p$=20%) that were $K_o$ consolidated in a slurry oedometer to 1.0 kg/cm$^2$ (with one cycle of secondary compression), rebounded to 0.25 kg/cm$^2$, trimmed from the large oedometer cell and then stored at constant water content for up to several months. Standard oedometer tests showed a well defined increase in $\sigma'_p$ with storage time due to thixotropy [O'Neill et al. (1985)] ($\sigma'_p$ for the samples shown ranged from 1.25 to 1.6 kg/cm$^2$). The data are normalized for simplicity to the $\sigma'_p$ corresponding to the age of the test specimen, although this involves some error.

Tests (1) through (4) in Fig.3 were isotropically consolidated to $\sigma'_{vc} \cong 0.25$ kg/cm$^2$ and then loaded incrementally with drainage to measure the behaviour of overconsolidated clay. Test (7) is also overconsolidated, but sheared undrained in triaxial compression. Tests (5) and (6) were $K_o$ consolidated well into the virgin compression range and sheared undrained in triaxial extension and compression, respectively, to measure the behaviour of normally consolidated clay. Yielding was assumed to occur at stresses causing deviation from the initial "linear" stress-strain curve or a change in the shape of the undrained effective stress path for Test (7).

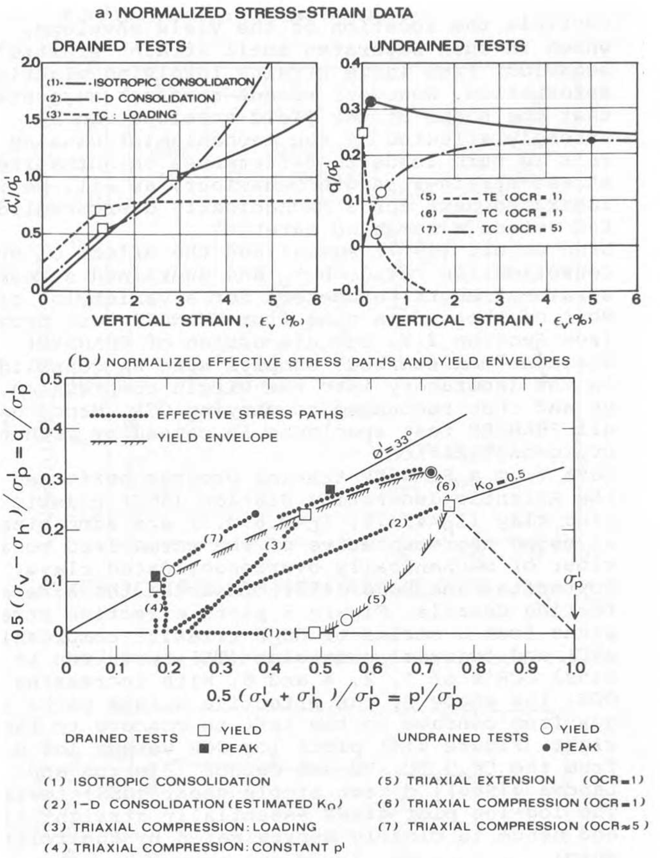



Fig.3: (a) Normalized Stress-Strain, (b) Effective Stress Paths and Yield Envelopes from Drained and Undrained Tests on Resedimented Boston Blue clay [Data from Bensari (1984) and O'Neill (1985)].

The results for resedimented Boston Blue clay show the following features:

1. The yield envelope is roughly centered about the $K_o$ line such that the yield stress for isotropic consolidation is significantly less than $\sigma'_p$. Both of these characteristics appear typical of soft sedimentary clay deposits; see the review by Tavenas and Leroueil (1977).

2. The effective stress path for the $K_o$ consolidated-undrained triaxial extension ($CK_oUE$) test runs on normally consolidated clay coincides with the lower portion of the yield envelope.

3. The upper portion of the yield envelope at high values of $p'/\sigma'_p$ coincides with the initial part of the effective stress path for the $K_o$ consolidated-undrained triaxial compression ($CK_oUC$) test run on normally consolidated clay. At lower values of $p'/\sigma'_p$, the envelope falls near the maximum obliquity Mohr-Coulomb envelope for normally consolidated clay.

4. As a reasonable approximation, the yield envelope for this clay can be defined from the effective stress paths for $CK_oU$ compression and extension tests on normally consolidated clay and the large strain $\phi'$ envelope. This hypothesis is considered reasonable for overconsolidated clays where mechanical precompression is the principal mechanism responsible for the preconsolidation pressure.

Results are now presented for the James Bay B-6 marine clay to illustrate very different behaviour for a material wherein cementation is thought to be the principal mechanism contributing to the measured preconsolidation pressure profile shown in Fig.2. This Canadian deposit is a very brittle, highly structured quick clay. The data are from consolidated-undrained triaxial compression and extension tests having the following consolidation conditions (all using $K_c$=0.55):

1. Consolidation to $\sigma'_{vc}=\sigma'_{vo}$ at five depths to measure the behaviour of "intact" clay at the in situ OCR, which varied from about 1.8 to 3.3.

2. Consolidation to $\sigma'_{vc}/\sigma'_p$ between 0.6 and 1.0 at two depths to measure the effect of recompression for intact clay (the volumetric strains were 2% or less).

3. Consolidation to $\sigma'_{vc}$ ranging from 1.3 to 3 times the in situ $\sigma'_p$ to measure the behaviour of "destructured" clay [Leroueil et al. (1979)].

Figure 4(a) plots stress-strain curves from three typical triaxial compression tests, where yielding essentially occurs at the peak strength (particularly for the highly structured intact clay). The normalized $q-\sigma'_p$ stresses at the peak strength for all triaxial compression tests and representative effective stress paths are plotted in Fig.4(b). Approximate locations of the yield envelopes are drawn based on the triaxial compression peak strengths and relevant effective stress paths. The results show the following important behavioral trends:

1. At the in situ OCR, the intact clay is extremely brittle, with approximately linear effective stress paths (A≅0.2 in compression and A≅0.8 in extension) prior to yielding. Further straining produces a very dramatic decrease in shear resistance, i.e. pronounced strain softening. Such behaviour is very different from that shown for a mechanically overconsolidated clay such as Test (7) in Fig.3.

2. Recompression to vertical stresses less than the in situ $\sigma'_p$ causes a modest increase in the peak strength and yield stress of the intact clay.

3. The upper portion of the yield envelope for intact clay (i.e. from samples having $\sigma'_{vc}$ less than the in situ $\sigma'_p$) rises well above the large strain Mohr-Coulomb envelope represented by the $\phi'$=35° line. This behaviour is also very different from that measured for resedimented Boston Blue clay.

4. Reconsolidation to $\sigma'_{vc}$ values well beyond the in situ $\sigma'_p$ produces effective stress paths and a corresponding yield envelope for destructured clay that are similar in shape to those measured for Boston Blue clay.

The above behaviour for intact versus destructured clay is very similar to that reported by Leroueil et al. (1979) for tests on the St. Alban clay, which is representative of the highly structured, sensitive Champlain sea clays of eastern Canada. The upper portion of the yield envelope for medium to soft clays which derive their preconsolidation pressure principally from cementation bonds (or similar effects) lies well above the large strain Mohr-Coulomb envelope and such materials are characterized by brittle behaviour prior to yielding during both undrained and drained loading. Substantial strain softening occurs after yielding for stress paths well above the $K_o$ line,

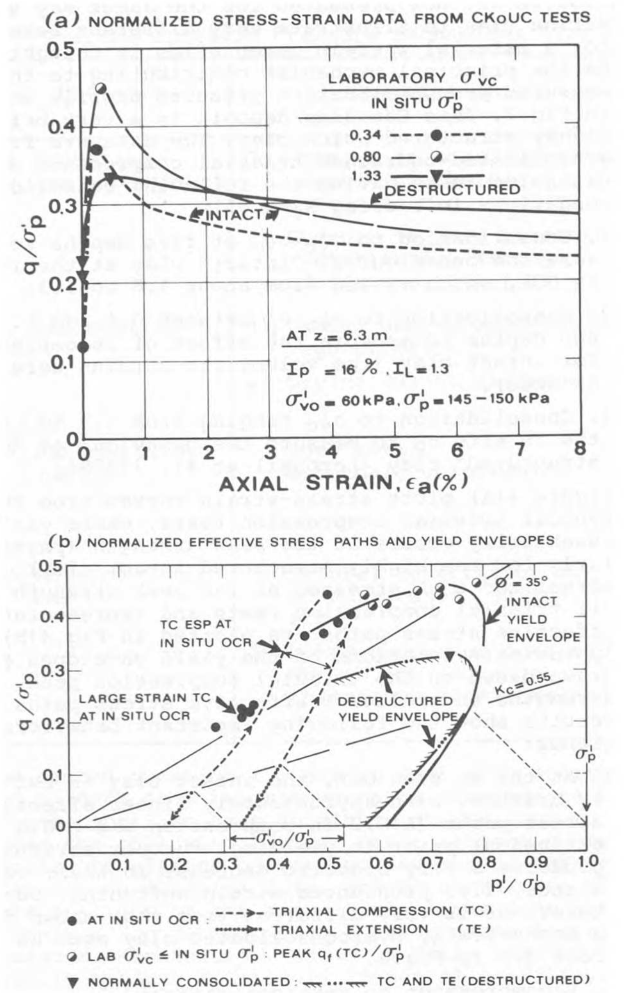



Fig.4: (a) Normalized Stress-Strain, (b) Effective Stress Paths and Approximate Yield Envelopes from $CK_oU$ Tests on James Bay B-6 Marine Clay [Data from Lefebvre et al. (1983)].

whereas compressibility increases dramatically for drained loading near and below the $K_o$ line. By comparison, mechanically overconsolidated clays have a more compact yield envelope and exhibit a much more ductile behaviour. Mechanical overconsolidation also increases the in situ $K_o$, whereas $K_o$ for cemented deposits may well remain near the normally consolidated value (Note: in situ measurement of $K_o$ in cemented materials is difficult since the soil is very brittle). Appropriate test procedures for minimizing the adverse effects of sample disturbance (Section 2.3.) also depend on the preconsolidation pressure mechanism(s).

2.2.4. <u>Normalization</u> (Normalized Behaviour and Use of Normalized Parameters)

It should be evident that the in situ preconsolidation pressure, $\sigma'_p$, is the single most important parameter controlling basic behavioral trends for a wide range of problems involving both undrained and drained load applications. It controls the location of the yield envelope, which in turn separates small strain "elastic" behaviour from large strains involving plastic deformation. However, recent research suggests that the shape of the yield envelope can be strongly affected by the mechanism(s) causing $\sigma'_p$. This in turn leads to differences in normalized stress-strain-strength behaviour, as will be illustrated next for a mechanically overconsolidated versus a cemented material.

Ladd et al. (1977) summarized the effect of overconsolidation ratio on $K_o$ and undrained stress-strain-strength parameters for a variety of clays. Most of those data came from SHANSEP test programs (see Section 2.3. for discussion of SHANSEP) wherein "undisturbed" samples were $K_o$ consolidated in the laboratory into the virgin compression range and then rebounded to varying OCR. Hence $\sigma'_p$ for all SHANSEP test specimens is caused by mechanical overconsolidation.

Data from a SHANSEP testing program performed on the Atlantic Generating Station (AGS) plastic marine clay ($I_P$=43±7%, $I_L$=0.6±0.1) are summarized as being representative of the normalized behaviour of mechanically overconsolidated clays. Koutsoftas and Ladd (1984) describe the site and testing details. Figure 5 plots effective stress paths from a series of $CK_oU$ triaxial compression (TC) and triaxial extension (TE) tests run at nominal OCR's of 1, 2, 4 and 8. With increasing OCR, the shape of the effective stress paths changes from concave to the left to concave to the right. Figure 6(a) plots log OCR versus log $c_u/\sigma'_{vc}$ from the $CK_oU$ TC, TE and Geonor [Bjerrum and Landva (1966)] direct simple shear (DSS) tests. The log-log plot gives essentially straight lines and hence is closely approximated by the relationship:

$$c_u/\sigma'_{vc} = S\ (OCR)^m$$

where S is the undrained strength ratio for normally consolidated (OCR=1) clay. Figure 6(b) shows the variation in shear strain and Skempton's pore pressure parameter at failure (peak strength). Figure 7 presents similar plots from undrained shear tests run on the cemented James Bay B-6 marine clay deposit (see Figs.2 and 4 for the stress history and typical effective stress paths respectively). For this test program, all overcon-

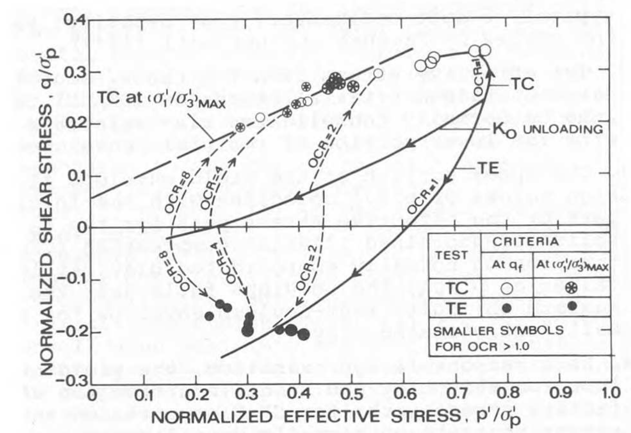



Fig.5: Normalized Effective Stress Paths from SHANSEP $CK_oU$ Tests on AGS CH Marine Clay [Koutsoftas and Ladd (1984)].

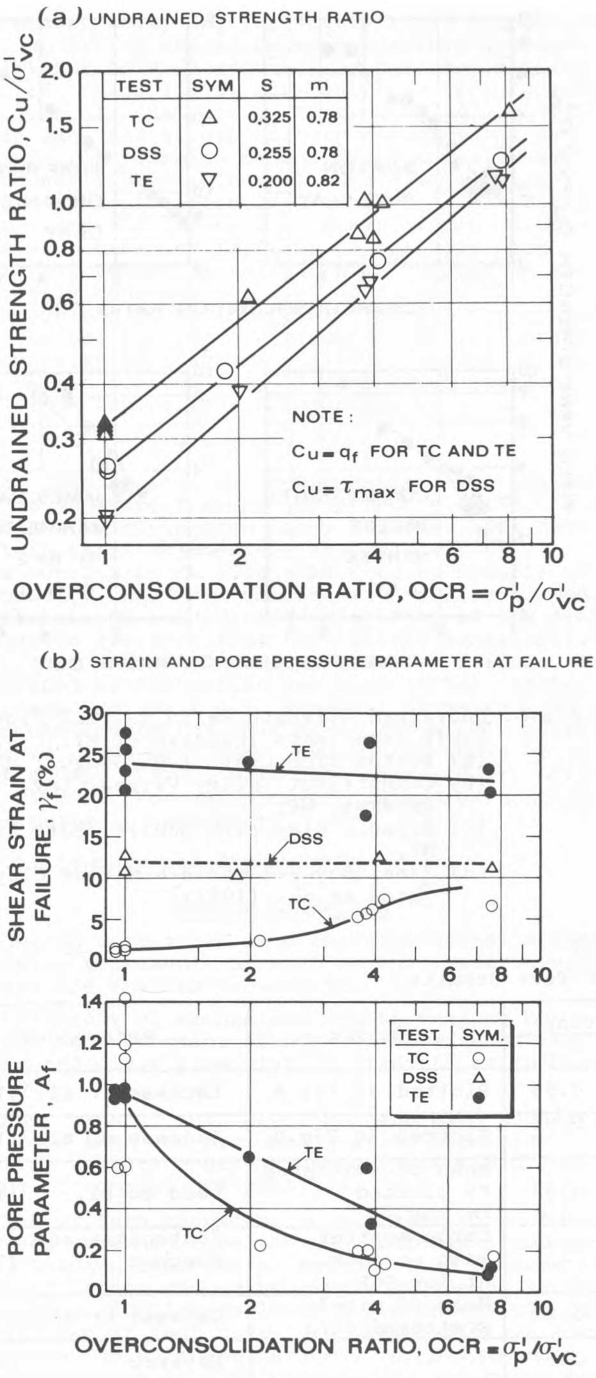



Fig.6: (a) Undrained Strength Ratio and (b) Strain and Pore Pressure Parameter A at Failure vs. OCR from SHANSEP $CK_0U$ Tests on AGS CH Marine Clay [Koutsoftas and Ladd (1984)].

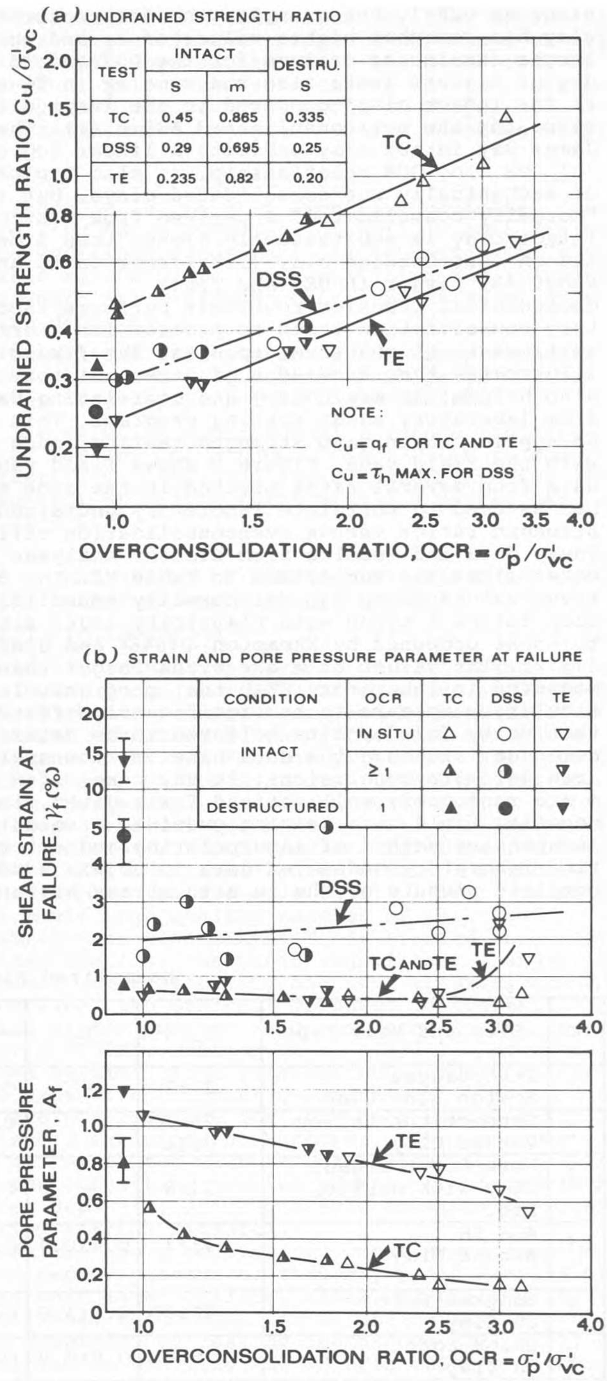



Fig.7: (a) Undrained Strength Ratio and (b) Strain and Pore Pressure Parameter A at Failure vs. OCR from $CK_0U$ Tests Run on Intact and Destructured James Bay B-6 Marine Clay [Data from Lefebvre et al. (1983)].

solidated data are from samples reconsolidated to stresses less than the in situ $\sigma'_p$, i.e. representing behaviour of an intact cemented clay. The OCR = 1 data on destructured clay had a laboratory $\sigma'_{vc}$ ranging from 1.3 to 3 times the in situ $\sigma'_p$.

As previously noted, the yield envelope for intact James Bay clay is much higher than for destructured clay, where the large strains caused by consolidation beyond the in situ $\sigma'_p$ presumably destroy most of the cementation bonds. Likewise, results in Fig.7 show a discontinuity in behaviour at OCR=1. For example, the destructured clay has somewhat higher values of $A_f$ and much larger strains at failure for the DSS and TE modes of failure (note also the same $\gamma_f$ in TC and TE for intact clay* compared to the large difference for the overconsolidated AGS clay). The James Bay intact clay exhibits a linear log $c_u/\sigma'_{vo}$ vs. log OCR relationship, as also typical of mechanically overconsolidated clays. But the "normally consolidated" S derived from testing intact clay is substantially higher than S measured on destructured clay (35% larger in TC and about 15% larger in DSS and TE).

Geotechnical practice routinely performs laboratory consolidation tests to predict long term settlements of cohesive deposits. The foregoing illustrates that knowledge of stress history is also helpful in evaluating and correlating data from laboratory shear testing programs. This also applies to in situ strength testing, such as with the field vane. Figure 8 shows field vane data from several sites plotted in the same manner as used to correlate laboratory undrained strength ratios versus overconsolidation ratio. These data and results from similar analyses at other sites are summarized in Table VI. The derived values of $c_u/\sigma'_{vo}$ for normally consolidated clay follow a trend with plasticity index similar to those proposed by Skempton (1948) and Bjerrum (1972). The values of m are often larger than measured in laboratory $CK_oU$ test programs. In particular, m appears to be significantly greater than unity for deposits believed to be naturally cemented, although the data base is too small to draw definite conclusions. In any case, if S and m are reasonably well defined for a given clay deposit, field vane testing provides a relatively inexpensive method of interpolating and extrapolating laboratory oedometer data to obtain a more complete picture of the in situ stress history.

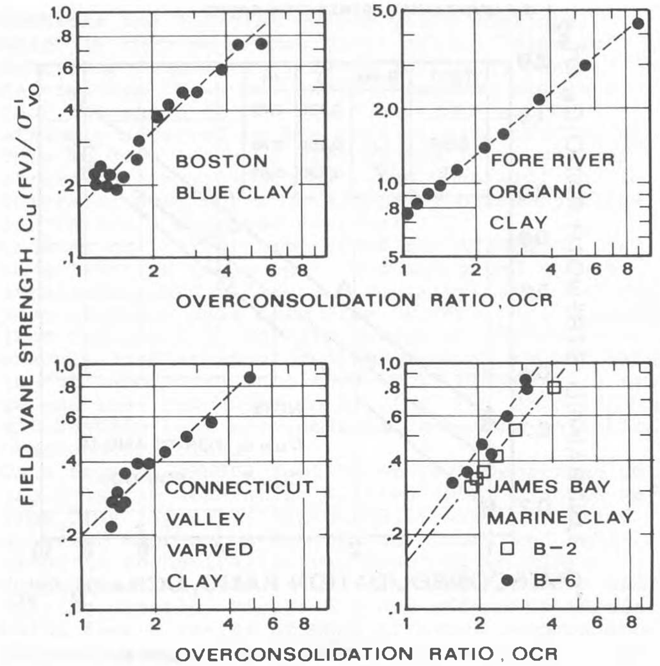



Fig.8: Undrained Strength Ratio vs. OCR from Field Vane Tests [Lacasse et al. (1978)];
(a) Boston Blue Clay, I-95 Saugus; MA
(b) Connecticut Valley Varved Clay, Amherst, MA;
(c) Organic Clay with Shells, Fore River, ME;
(d) James Bay B-2 and B-6 Marine Clays [Ladd et al. (1983)].

TABLE VI

Normalized Field Vane Test Results

| No. | Site and Soil Type | $I_p$ (%) | $I_L$ | Field Vane S | Field Vane m | Remarks | References |
|---|---|---|---|---|---|---|---|
| 1 | I-95 Saugus Boston Blue Clay | 21±3 | - | 0.165 | 0.96 | Plotted in Fig.8 | Lacasse et al. (1978) |
| 2 | Connecticut Valley Varved Clay | 25-30 Bulk | - | 0.20 | 0.93 | Plotted in Fig.8 | Lacasse et al. (1978) |
| 3 | Fore River Organic Clay with Shells, Etc. | 34±8 | - | 0.74 | 0.83 | Mean of scattered FV plotted in Fig.8 | Ladd et al. (1969) |
| 4 | AGS CH Marine Clay | 43±7 | 0.6±0.1 | 0.33 | 0.77 | Large scatter with no data at low OCR | Koutsoftas and Fischer (1976) |
| 5 | Bangkok Soft CH Clay | 41±20 | 0.7±1.0 | 0.29 | 0.80 | Mean of highly scattered data | Lacasse et al. (1978) |
| 6 | Omaha Soft CH Clay | 50 | 0.8±0.9 | 0.28 | 0.97 | - | Navarro (1984) |
| 7 | New Liskeard Cemented Varved Clay | 25±5 Bulk | - | 0.185 | 1.51 | Uncertainty in OCR | Lacasse et al. (1978) |
| 8 | James Bay B-2 Marine Clay | 9.5±3 | 2.6±0.9 | 0.16 | 1.18 | Mean FV from 8 profiles Plotted in Fig.8 | Ladd et al. (1983) |
| 9 | James Bay B-6 Marine Clay | 13±3 | 1.9±0.6 | 0.17 | 1.35 | Mean FV from 5 profiles Plotted in Fig.8 | |

* The larger DSS $\gamma_f$ for intact clay might have been partly due to the use of top and bottom pins.

For m near unity and relatively small changes in OCR, $c_u(FV)/\sigma'_p$ should be approximately constant ($\pm$ 15% for $\Delta OCR = 2$ and $m = 1.0 \pm 0.2$) for a given deposit. Mesri (1975) substantiated this "prediction" in his analysis of Bjerrum's (1972, 1973) field vane and stress history correlations developed for normally consolidated and slightly overconsolidated (OCR < 2) post glacial clays. Moreover, application of Bjerrum's (1972) field vane correction factor, $\mu$, based on case histories of embankment failures to the resulting $c_u(FV)/\sigma'_p$ versus $I_p$ relationship yielded an essentially constant $\mu\ c_u(FV)/\sigma'_p = c_u/\sigma'_p = 0.22$. Although the above interpretation did not consider the scatter about Bjerrum's (1972) $\mu$ versus $I_p$ correlation, Larsson's (1980) summary of $c_u/\sigma'_p$ values derived from 15 embankment failures for clays having $I_p$ < 60% yields:

$$c_u/\sigma'_p = 0.23 \pm 0.04$$

Hence, the in situ $c_u/\sigma'_p$ appropriate for analyses of embankment stability probably falls within a fairly narrow range for most soft sedimentary clays of moderate to low plasticity. This conclusion is also supported by results of laboratory $CK_oU$ tests sheared with differing modes of failure to incorporate anisotropy if the peak strengths are corrected for "strain compatibility", i.e. the effects of progressive failure, as described by Koutsoftas and Ladd (1984) (Note: also see Section 2.4.). However, there is some evidence indicating that $c_u/\sigma'_p$ for highly plastic organic clays is higher than quoted above [e.g. Trak et al. (1980), Holtz and Holm (1979), Larsson (1980)]. For example, the ratio for nonfiberous peats in very high.

#### 2.2.5. Summary and Conclusions

**Preconsolidation**

1. The $\sigma'_p$ determined from one-dimensional drained loading represents a point on the yield envelope where the $K_o$-line crosses it.
2. Different $\sigma'_p$ mechanisms lead to varying behaviour as a function of OCR. Some evidence that cemented clays give high FV strength as function of OCR.
3. Many natural soil deposits have been subjected to more than one preconsolidation mechanism, acting simultaneously or sequentially and leading to complex OCR profiles.
4. The writers recommend to evaluate $\sigma'_p$ on EOP curves obtained from the conventional incremental loading oedometers. Reduced load increments (=0.5) ratios should be used when the $\sigma'_p$ is approached. Continuous loading oedometer (CRS, CHG) tests run at fast rates can overpredict in situ $\sigma'_p$. When using these devices, the strain rate corresponding to the conventional EOP curve may be inferred from the formula given in section 2.5.2.

**Yield Envelope**

1. The shape of the yield envelopes of resedimented mechanically OC Boston Blue clay and cemented sensitive James Bay clay are very different. This difference is attributed to the very different preconsolidation mechanisms experienced by these two soils. The writers emphasize the need for further research on the relation between $\sigma'_p$ mechanisms and yield mechanisms for a variety of natural clays.
2. Consolidation of the James Bay clay beyond $\sigma'_p$ causes a significant change in the shape of its yield envelope and hence in its mechanical behaviour. This phenomenon, called "destructuration", is typical for highly structured clays, e.g. overconsolidated cemented eastern Canadian clays.

**Normalization**

1. Since $\sigma'_p$ is the most important parameter controlling basic behaviour of soil in both drained and undrained conditions, it is particularly suitable as the basis for normalizing the relevant strength and stiffness characteristics of cohesive deposits.
2. Normalized strength and stiffness parameters do not lead to unique relationships vs OCR because they depend on the $\sigma'_p$ mechanism.
3. The existing experimental evidence indicates that for low OCR inorganic clays of low to moderate plasticity, the $c_u/\sigma'_p$ ratio appropriate for embankment stability falls within a narrow band of $0.23 \pm 0.04$.

### 2.3. ASSESSMENT OF SAMPLE DISTURBANCE AND PROCEDURES TO MINIMIZE ITS EFFECTS

#### 2.3.1. Introduction

Table VII summarizes sources of disturbance which can occur during sampling of cohesive soils from a drill hole. Some of these can be controlled or at least minimized by using the best available sampling technique for the particular soil conditions and by following appropriate handling procedures. For example, researchers in Eastern Canada developed special techniques [e.g. the Sherbrooke 250 mm diameter block sampler, Lefebvre and Poulin (1979), and the Laval 200 mm diameter overcored tube sampler, La Rochelle et al. (1981)] to obtain extremely high quality samples of their soft brittle sensitive clays. Their studies show that better sampling can yield significant changes in drained compressibility and peak triaxial strengths (undrained and drained) compared to conventional fixed piston samples [e.g. La Rochelle and Lefebvre (1970), Raymond et al. (1971)]. However, these special large diameter samplers are either too expensive for most geotechnical investigations or are not feasible for very deep sampling and for offshore exploration. Moreover, other sources like those associated with stress relief, and especially the outgassing problem often encountered in deep water deposits, are impossible to minimize without taking extreme measures.

Given the above constraints, most sampling programs must employ procedures that may yield samples of less than ideal quality. Hence, practicing engineers need techniques for assessing sample quality and knowledge of testing techniques that might be employed to minimize the adverse effects of sample disturbance. Our treatment of these two subjects will be restricted to cohesive soils where design practice relies heavily on laboratory testing. However, it should be mentioned that recent progress has been made in developing procedures to obtain much better quality samples of granular soils, [e.g. Marcuson and Franklin (1979)].

#### 2.3.2. Sample Quality (Cohesive Soils)

Three techniques for assessing sample quality will be discussed: radiography; measurements of effective stress; and evaluating oedometer compression curves.

TABLE VII

Sources of Sample Disturbance in Cohesive Soils

| Heading | Item | Remarks |
|---|---|---|
| 1. Stress Relief | 1.1. Change in stresses due to drilling hole | . Excessive reduction in $\sigma_v$ due to light drilling mud causes excessive deformations in extension |
| | | . Overpressure causes excessive deformations in compression |
| | 1.2. Eventual removal of in situ shear stress | . Resultant shear strain should usually be small |
| | 1.3. Eventual reduction (removal) of confining stress | . Loss of negative u (soil suction) due to presence of coarser grained materials |
| | | . Expansion of gas (bubbles and/or dissolved gas) |
| 2. Sampling Technique | 2.1. Sampler geometry: Diameter/Length Area ratio Clearance ratio Accessories -- piston, coring tube, inner foil, etc. | These variables affect: . Recovery ratio . Adhesion along sample walls . Thickness of remolded zone along interior wall |
| | 2.2. Method of advancing sampler | . Continuous pushing better than hammering |
| | 2.3. Method of extraction | . To reduce suction effect at bottom of sample, use vacuum breaker |
| 3. Handing procedures | 3.1. Transportation | . Avoid shocks, changes in temperature, etc. |
| | 3.2. Storage | . Best to store at in situ temperature to minimize bacteria growth, etc. |
| | | . Avoid chemical reactions with sampling tube |
| | | . Opportunity for water migration increases with storage time |
| | 3.3. Extrusion, trimming, etc. | . Minimize further straining |

## Radiography

X-ray photons emitted from a cathode ray tube penetrate materials to varying degrees depending upon their density. A radiograph is the photographic record produced by the passage of X-rays through an object onto a white photographic film that darkens in proportion to the intensity of protons reaching it. Photographs taken of radiographs produce the opposite effect. Hence, a low density object appears as a dark zone on a radiography (negative) and as a light zone on a photograph (positive). Although industrial radiography has been used for many years in geotechnical research, primarily in England for measurement of strains via lead shot embedded in soil samples (Arthur, 1972), its systematic use with tube samples has only recently become fairly widespread. Based on experience gained at MIT since 1978, radiography can show the following:

1. Variations in soil types, especially granular versus cohesive materials.
2. Macrofabric features resulting from bedding planes, varves, fissures, shear planes, etc.
3. Presence of "intrusions" such as sand lenses, stones, shells, calcareous nodules, peaty materials, drilling mud, etc.
4. Voids and cracks due to gas pockets.
5. Variations in the degree of sample disturbance, ranging from barely detectable curvature adjacent to the sample edges to gross disturbance as evidenced by a completely contorted appearance and large voids and cracks (most often occurring at the ends of the tube).

Many of these features may not be readily identified from visual inspection of the extruded samples, at least without trimming or breaking it apart. Hence radiography provides a nondestructive means for selecting the most representative and/or less disturbed portions of each tube for engineering tests. It also helps in planning the overall testing program based on the amounts of suitable material. Such information can be considered essential for projects having a limited number of expensive tube samples, as occurs with offshore exploration. In fact, the Norwegian Geotechnical Institute used onboard radiography in 1983 to provide an immediate assessment of sample quality at a deep water (300 m) site in the North Sea [Lacasse et al. (1984)].

The American Society for Testing and Materials is currently developing an ASTM Standard for radiographing soil samples. The writers predict that radiography facilities will soon become standard on drilling ships and in geographic areas having a significant number of organizations engaged in geotechnical engineering. The expense of radiography is relatively small (approximately US $ 50 per tube) compared to the cost of running consolidation tests and sophisticated strength tests on disturbed and/or nonrepresentative soil specimens.

## Effective Stress After Sampling

"Perfect sampling" denotes no disturbance other than that caused by the undrained removal of the in situ shear stress, $\Delta q = 0.5\ \sigma'_{vo}\ (1 - K_o)$. Assuming complete saturation, the isotropic effective stress, $\sigma'_{ps}$, after perfect sampling is given by [Ladd and Lambe (1963)]:

$$\sigma'_{ps} = \sigma'_{vo}\ [K_o + A_u\ (1 - K_o)]$$

where $A_u = (\Delta u - \Delta\sigma_h) / (\Delta\sigma_v - \Delta\sigma_h)$. For practical purposes, $A_u$ is quite small for normally consolidated clays and hence $\sigma'_{ps}$ is approximately equal to $\sigma'_{ho} = K_o\ \sigma'_{vo}$.

The various sources of disturbance listed in Table VII will decrease the effective stress of laboratory specimens $\sigma'_s$ to values less than $\sigma'_{ps}$. The resultant magnitude of $\sigma'_s$ can be measured in a triaxial cell fitted with a fine porous stone at the base ($\sigma'_s$ = cell pressure - pore pressure) or by inserting a small pore pressure probe into an unconfined sample. Data collected at MIT shows that $\sigma'_s/\sigma'_{ps}$ typically equal 0.2 ± 0.2 for conventional tube sampling of medium to soft clays of moderate to low sensitivity at depths greater than 5 to 10 m. The decrease in effective stress from $\sigma'_{ps}$ to $\sigma'_s$ can occur at constant water content due to mechanical disturbance since shear deformations in low OCR clays produce positive excess pore pressures. The decrease can also result from swelling as water is sucked from: (1) any cohesionless zones that cannot sustain negative pore pressures; (2) more disturbed zones, such as the remolded (more compressible) layer along the walls of the tube. Internal swelling has been observed to decrease $\sigma'_s$ [Schjetne (1971)] and even to reduce the strength of Norwegian clays measured in $CK_oU$ triaxial compression tests reconsolidated to the overburden stress [Bjerrum (1973)]. However, La Rochelle et al. (1976) conclude that it is apparently less significant in strongly cemented clays.

Several researchers have proposed that $\sigma'_s/\sigma'_{ps}$ provides a quantitative measure of the degree of sample disturbance [e.g. Ladd and Lambe (1963), Nelson et al. (1971), Okumura (1971)], and it can serve as a useful guide regarding likely decreases in measured UU strengths. But no data are available to correlate $\sigma'_s/\sigma'_{ps}$ with the effects of disturbance on CU or oedometer test results. Moreover, $\sigma'_s/\sigma'_{ps}$ has little relevance in highly cemented clays since $\sigma'_s$ can approach zero without seriously affecting the structure of these soils.

## Oedometer Curves

Sample disturbance usually affects compression curves from oedometer tests run on "ordinary" sedimentary clays (soft to stiff consistency, low to moderate sensitivity) in the following manner:

1. Decreases the void ratio (or increases the strain) at any given consolidation stress, $\sigma'_{vc}$.
2. Makes it more difficult to define the point of minimum radius, thus obscuring and often lowering the value of $\sigma'_p$.
3. Increases the compressibility during recompression (always true) and may decrease the compressibility in the virgin compression region.

Figure 9(a) illustrates the above effects via data from oedometer tests run on specimens from the same tube of an offshore Holocene clay. Although Test A gave a reasonable looking curve that indi-

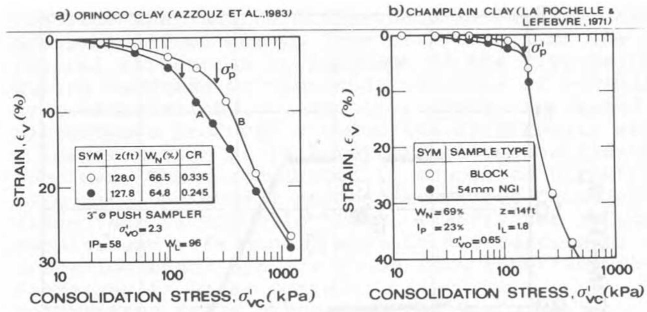



Fig.9: Effects of Disturbance on Oedometer Compression Curves: (a) Orinoco Clay [Azzouz et al. (1982)]; (b) Champlain Clay, St. Louis [La Rochelle and Lefebvre (1971)].

cates an underconsolidated clay, the second test yielded a significantly higher $\sigma'_p$ which agrees with the stress history developed for the deposit. Test B also had a much higher virgin compressibility.

Figure 9(b) compares compression curves from block and tube samples of a brittle, sensitive clay, where disturbance increases compressibility during recompression but has little effect on $\sigma'_p$ and virgin compressibility. These results appear fairly typical of the cemented Champlain Sea clays where research [see La Rochelle et al. (1981), for references] shows that very large diameter sampling is required to preserve the stress-strain properties of overconsolidated intact clay (i.e. within the yield envelope shown in Fig.4). Conventional fixed piston samples may or may not affect the location and shape of the virgin curve depending on the tube diameter and specific deposit.

The above discussion and examples demonstrate that no simple criteria exist to evaluate sample quality from compression curves. Whether or not sample disturbance is significant also depends on the properties being assessed, e.g. overconsolidated versus virgin behaviour, $\sigma'_p$, etc. Nevertheless, a comparative evaluation of compression curves within a given deposit can often indicate different degrees of disturbance from relative changes in compressibility and shapes of the curves. One simple, but not precise, criteria is the measured vertical strain at the effective overburden stress for deposits believed to have a relatively uniform stress history, as illustrated in Fig.10 for a Holocene deposit offshore eastern Venezuela.

## Conclusions

1. The required sample quality depends on what properties have to be measured and their required degree of accuracy. This can vary from:
   - Large diameter specially cored samples to measure the intact properties of brittle clays.
   - More conventional push tubes (preferably fixed piston) for routine settlement and stability problems.
2. Quantitative assessment of sample quality is not easy. The task is almost impossible if accurate small strain behaviour is needed unless block samples for comparison are available. But for routine settlement and stability problems involving relatively ductile materials, qualitative assessment can be obtained via:
   - Radiography-ideally suited to help select representative and best quality materials and to plan overall program based on available suitable soil.

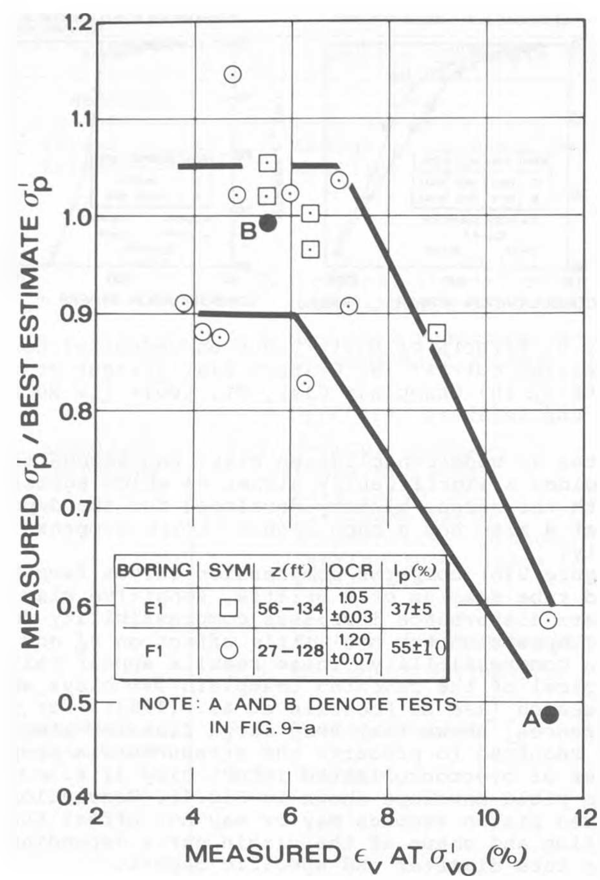



Fig.10: Recompression Strain vs. Preconsolidation Pressure from Oedometer Tests in the Soft Orinoco Clay [from Ladd et al. (1980)].

- Measurements of sample effective stress, $\sigma'_s$, for comparison with that of perfect sampling, $\sigma'_{ps} \cong K_o\ \sigma'_{vo}$.
- Shape of and compressibility parameters from oedometer compression curve.

3. Conventional tube sampling will generally cause sufficient and variable disturbance such that UU type strength tests will usually not give reliable or repeatable stress-strain-strength data (also there are problems due to effects of strain rate and anisotropy). The profession is increasingly realizing this and hence there now is more emphasis on using results from samples reconsolidated in the laboratory. But what procedure should be employed?

### 2.3.3. Laboratory Reconsolidation Procedures to Minimize Effects of Sample Disturbance

#### One Dimensional Settlement

Even modest degrees of sample disturbance will generally increase compressibility during recompression by several fold. For ordinary clays, this error can be minimized by using the recompression curves from an unload-reload cycle performed after consolidating the sample beyond the in situ preconsolidation pressure. For highly structured and cemented materials, this technique is likely to overestimate the in situ recompression curve and it may be preferable to unload from a stress less than $\sigma'_p$ (the writers are not sure of this recommendation).

The influence of disturbance on virgin compressibility varies considerably with the degree of disturbance and soil type. For ordinary clays, Schmertmann's (1955) technique for reconstructing the in situ compression curve is recommended, this typically increasing $CR = C_c / (1 + e_o)$ by 15 ± 5% for moderately good samples of medium to soft clay [Ladd (1973)]. But with higly structured clays (high $S_t$ and $I_L$) having an extremely high and variable compressibility just beyond $\sigma'_p$, such as shown in Fig.9(b), one can only hope that the samples are of sufficient quality to preserve this characteristic.

#### Undrained Stress-Strain-Strength Properties

The in situ soil structure will always be altered by the sampling process and hence can never be exactly duplicated in the laboratory. It is now recognized that unconsolidated-undrained (UU) type testing produces highly unreliable and variable results for at least two reasons: (1) varying degrees of disturbance, which often cause a substantial reduction in the preshear effective stress, $\sigma'_s$; (2) even "perfect sampling" significantly alters stress-strain characteristics since shearing starts from isotropic rather than the in situ $K_o$ stress conditions [Ladd and Lambe (1964), Skempton and Sowa (1963), Noorany and Seed (1965)]. Isotropically consolidated-undrained (CIU) testing also has similar drawbacks and hence $K_o$ consolidated-undrained ($CK_oU$) tests are required.

Variables to be considered in conducting the consolidation portion of a $CK_oU$ test include the preshear values of: (1) the vertical consolidation stress, $\sigma'_{vc}$; (2) the consolidation stress ratio, $K_c = \sigma'_{hc} / \sigma'_{vc}$ and (3) the time allowed for consolidation, $t_c$; and the stress path and consolidation times used to reach the preshear condition. The following discussion will focus on the first variable, in particular the relative merits of the so-called Recompression and SHANSEP reconsolidation techniques which are used in many laboratories to minimize the adverse effects of sample disturbance.

These two techniques are illustrated in Fig. 11, which shows hypothetical in situ and laboratory $K_o$ compression curves for a slightly overconsolidated soft clay. Points 1 and 2 designate the in situ condition and the preshear effective stress for UU test respectively (the latter assuming no change in water content during sampling). In the Recompression technique, the test specimen is reconsolidated (ideally along a $K_o$ stress path) to $\sigma'_{vc}=\sigma'_{vo}$ shown by Point 3. Points A through D correspond to typical stresses used for a SHANSEP test program (to be subsequently described).

#### Recompression Technique

Bjerrum (1973) presents the rationale underlying this technique, which has since been routinely used by the Norwegian Geotechnical Institute (NGI) to predict in situ behaviour. The following selected quotes attempt to reflect his views (note that Bjerrum was then more concerned with internal swelling than gross mechanical disturbance).

"Provided the swelling which occurred before the testing is so small that it still is of an

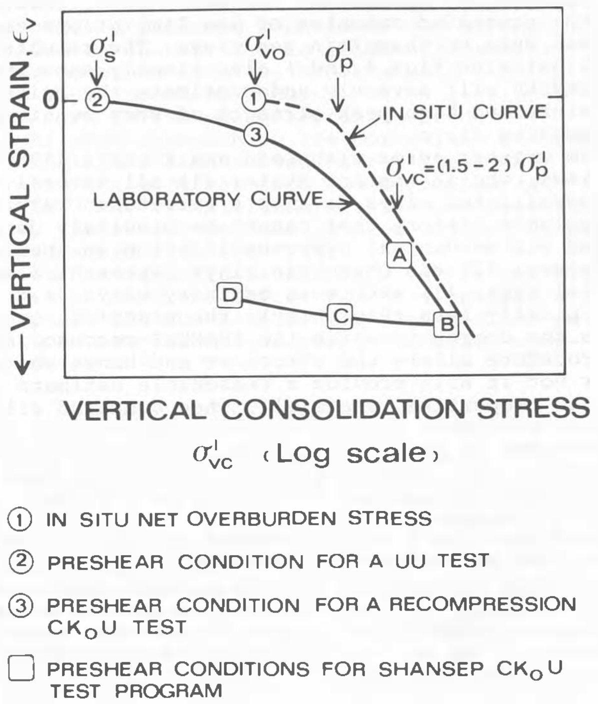



Fig.11: Consolidation Procedures for Laboratory $CK_oU$ Testing [After Ladd et al. (1977)].

'elastic' nature, and provided the mechanical disturbance is relatively small, the detrimental effect of the swelling can be eliminated if the sample is reconsolidated at exactly the same pressures as it carried in the field before it is tested". The beneficial effects are firstly "to replace the field stresses with an identical set of effective stresses in the laboratory", and secondly "the additional water adsorbed since sampling is squeezed out of the sample again".

Bjerrum illustrated the unreliable nature of UU stress-strain data and concluded by stating that reconsolidation to the field stresses is "a necessary condition for the reliable measurement of the true properties of undisturbed samples of soft clays" and that an appreciation of this "is considered to be one of the most significant advances in recent years..."
In a companion paper, Berre and Bjerrum (1973) present results from Recompression $CK_oU$ tests run on 95 mm fixed piston samples of seven low OCR (< 1.8) clays. The authors report reconsolidation volumetric strains to increase "from 1.6% in the most plastic clay to 2-4% in clays of low plasticity, which indicates that samples of clays of low plasticity are somewhat more disturbed than samples of highly plastic clays". They also state: "Provided the reduction in water content resulting from the consolidation at the field stresses is not too large, the results of a shear test with such a sample should give a fair presentation of the behaviour of the clay in the field". They acknowledge that, in principle, a reduced water content as a result of sampling disturbances would be accompanied by a gain in strength, but conclude that sample disturbance in fact has the opposite effect due to "the destruction of the original structural arrangement of the clay gained during hundreds or thousands of years of secondary consolidation...". In other words, increased disturbance produces a reduction in strength and a larger strain at failure for the reconsolidated specimen. Their conclusion is supported by extensive data on several highly structured Canadian clays [see La Rochelle et al. (1981), for typical results and references] wherein destruction of cementation bonds appears to be more important than decreases in water content.
The writers fully agree that the Recompression technique should be used to predict the in situ behaviour of highly structured clays characterized by a high sensitivity and liquidity index (say $S_t$ > 5-10 and $I_L$ > 1-1.5) and cemented soils. Accurate small strain behaviour for Canadian type clays probably requires very large diameter samples, although the increased sample disturbance with smaller samples will most likely lead to a reduced stiffness and undrained strength, i.e. conservative results. The Recompression technique is also recommended for highly overconsolidated stiff clays and within weathered crusts, where sample disturbance is likely to be less of a problem.
On the other hand, for tube samples of truly normally consolidated deposits, there can be little question that reconsolidation to $\sigma'_{vo}$ will usually produce a significant decrease in volume and hence undrained strengths that are too high*. [Note: isotropic consolidation to $\sigma'_{vo}$ will certainly increase the error]. Although one seldom encounters OCR=1 deposits on land (waste disposal facilities being a prime exception), they are found offshore in areas near major rivers. There are also many relatively unstructured natural deposits having a sufficiently low OCR that reconsolidation to point 3 in Fig.11 might lead to overestimates of strength with increasing sample disturbance. Moreover, there is no way of assessing whether Recompression errs on the safe or unsafe side, except perhaps by testing samples with varying degrees of disturbance.
Other than the unknown potential adverse effects of sample disturbance, the main uncertainty in using the Recompression technique lies in selecting an appropriate $K_c$. Ideally it should equal the in situ $K_o$, which is difficult to estimate for preconsolidation mechanisms other than mechanical. One-dimensional recompression will not give the in situ $K_o$ due to disturbance and the dependence of $K_o$ on loading versus unloading. Some laboratories like NGI use oedometer tests to measure $\sigma'_p$ and Brooker and Ireland's (1965) empirical correlation with OCR to estimate $K_o$ [Berre (1981)]. NGI also first isotropically consolidates the sample to $\sigma'_{hc}$ and then increases $\sigma'_{vc}$ to $\sigma'_{vo}$ (for $K_o$ < 1)**. The Technical University of Turin (TUT) measures $K_o$ versus OCR for the particular clay using the SHANSEP technique, which also strictly applies only to mechanical overconsolidation. Eastern Canadian

* The writers believe that overestimates of $c_u$ will usually be larger than reported by Kirkpartrick and Khan (1984) from their investigation of sampling effects with normally consolidated resedimented kaolin and illite.

** Kirkpatrick and Khan (1984) show little error from doing this compared to the more time consuming loading in small increments (but their results are for resedimented kaolin and illite).

laboratories simply select a $K_c$ of about 0.5 to 0.6 considered reasonable for their cemented deposits.

Except for problems in selecting $K_c$, the Recompression technique is relatively simple to execute and it directly gives stress-strain-strength data at each depth of testing. Potential errors in $K_c$ are probably not significant compared to uncertainties in the effects of sample disturbance. The latter requires more research of the type carried out in eastern Canada. In any case, the writers recommend that this technique should always be accompanied by detailed measurements of the stress history of the deposit in order to:

1. help estimate $K_o$;
2. interpolate and extrapolate the "point data" via correlations with $\sigma'_p$ and/or OCR;
3. use this correlation to check the reasonableness of the results compared to normalized stress-strain-strength data on similar clay deposits.

## SHANSEP Technique

SHANSEP is an acronym for Stress History and Normalized Soil Engineering Properties. As described by Ladd and Foot (1974) and Ladd et al. (1977), the SHANSEP design procedure for estimating the in situ undrained properties of a clay deposit involves the following basic steps (for any given "uniform" layer and required mode of failure):

1. Establish the stress history, i.e. the profiles of $\sigma'_{vo}$ and $\sigma'_p$, which determines the range of OCR values for which data are required.
2. Perform a series of $CK_oU$ shear tests on specimens consolidated beyond the in situ preconsolidation pressures (to $\sigma'_{vc}$ greater than 1.5-2 times $\sigma'_p$) to measure the behaviour of normally consolidated clay and also on specimens rebounded to varying OCR to measure overconsolidated behaviour.
3. Express the results in terms of normalized soil parameters (NSP) and establish NSP vs. OCR relationships, e.g. $c_u/\sigma'_{vc}$ vs. OCR as in Fig. 6.
4. Use these NSP relationships and the stress history information to compute profiles of $c_u$,etc.

Although SHANSEP was originally developed based on the empirical observation that it yielded reasonable results, the rationale for the reconsolidation technique used to minimize the effects of sample disturbance was predicated on the assumption that natural clays exhibit normalized behaviour. Referring to Fig.11, laboratory compression curves typically approach the in situ virgin compression curve when $\sigma'_{vc}$ exceeds about 1.5 to 2 times $\sigma'_p$. Thus test specimens A and B should have a structure similar to in situ normally consolidated clay and hence yield reasonably NSP. Likewise, tests C and D give data on samples having a well defined overconsolidation ratio.

Ladd and Foott (1974) state that: "The possibility exists that (the above reconsolidation technique) will destroy some important aspect of soil structure that has developed during and after formation of the clay deposit". They also note: "... this is clearly true with highly structured 'quick' clays and with naturally cemented deposits". But severe critics of SHANSEP like Mesri (1975) state that: "all natural clays possess a structure that is destroyed ..." and its reconsolidation technique "does not overcome sample disturbance; on the contrary if further remolds and disturbs the natural structure of the clay". This opinion was endorsed by Tavenas and Leroueil (1977, 1980), who presented examples of its limitations using test data on Champlain sea clays. The results illustrated Figs.4 and 7 also clearly show that SHANSEP will severely underestimate the brittle nature and high peak strength of very sensitive cemented clays.

The writers agree with Ladd and Foott's (1980) views, who in essence state: (1) all natural overconsolidated clays develop a structure over their geologic history that cannot be precisely duplicated via mechanical overconsolidation in the laboratory; (2) the Champlain clays represent a special case; (3) with more ordinary clays (e.g. $I_L$ typically less than unity), the practical question is the degree to which the SHANSEP reconsolidation procedure alters the structure and hence whether or not it will provide a reasonable estimate of in situ properties, especially when compared with other available techniques; (4) quantitative assessment of potential errors are difficult to measure in the laboratory since it requires testing natural clay samples that truly retain their in situ structure.

Ladd and Foot (1980) presented the results in Fig.12, which compares undrained stress-strain triaxial compression data on a block sample of Connecticut Valley varved clay using the Recompression and SHANSEP techniques ($K_o$ assumed equal to unity at the in situ OCR of 4). The agreement is considered reasonable for engineering practice.

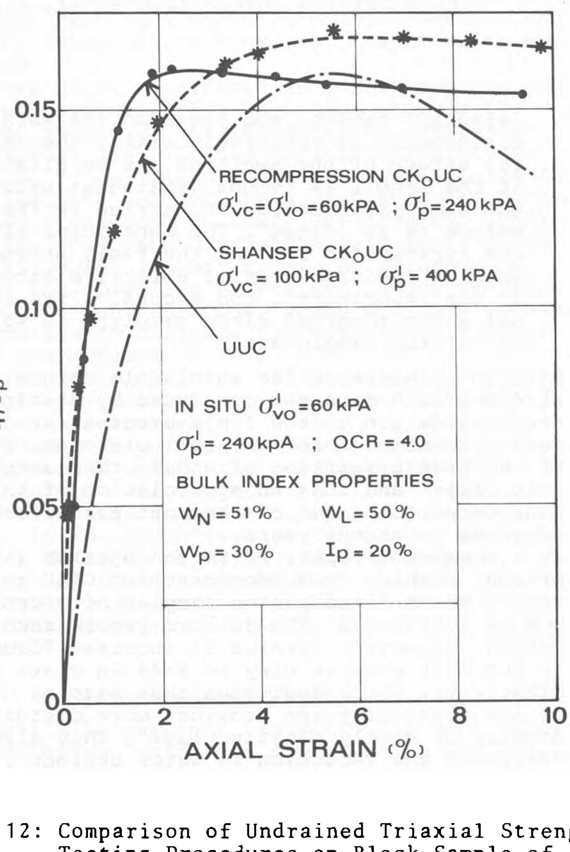



Fig.12: Comparison of Undrained Triaxial Strength Testing Procedures on Block Sample of Connecticut Valley Varved Clay [After Sambhandharaska (1977)].

Moreover, SHANSEP tests on tube samples, where disturbance produced much lower UUC strengths, yielded $c_u/\sigma'_{vc}$ vs. OCR relationships similar to those obtained with the block sample (though less true regarding modulus, which is mich more sensitive to the exact testing procedure). Unpublished research by the Norwegian Geotechnical Institute also showed nearly identical stress-strain curves from Recompression and SHANSEP $CK_oU$ triaxial compression and extension tests run on 200 mm fixed piston samples of plastic Drammen clay (OCR=1.5).
Similar research is needed on a variety of clay deposits. However, before reaching general conclusions, the research should also include results on samples having varying degrees of disturbance.

#### 2.3.4. Summary and Conclusions

The Recompression technique:

1. Is clearly superior for highly structured high $S_t$ and $I_L$ deposits, such as typical of eastern Canada, where accurate small strain behaviour may also require block quality samples.
2. Is preferred whenever block quality samples are available and for testing weathered and highly overconsolidated deposits where SHANSEP is difficult to apply.
3. As a result of sample disturbance, can underestimate $c_u$ in overconsolidated cemented deposits (well documented for Champlain clays) and may overestimate $c_u$ in normally to lightly overconsolidated "ordinary" clays (more an opinion than proven fact).
4. Requires an estimate of the in situ $K_o$, although resulting errors are probably not significant compared to potential adverse effects of disturbance.
5. Should always be accompanied by measurements of the in situ stress history in order to (1) estimate $K_o$; (2) extrapolate and interpolate the "point" data; and (3) check the reasonableness of the measured $c_u/\sigma'_{vo}$ values.

The SHANSEP technique:

1. Requires a more accurate estimate of the in situ stress history and is strictly applicable only to mechanically overconsolidated and truly normally consolidated deposits having ideal normalized behaviour.
2. Involves more extensive (and expensive) testing, especially regarding $CK_oU$ triaxial (or plane strain) tests which should closely follow a $K_o$ stress path. Special cells such as described by Campanella and Vaid (1972) or Bishop and Wesley (1975) greatly simplify this task.
3. Is preferred for testing tube samples from relatively deep deposits of low OCR "ordinary" clays (again more an opinion than proven fact).
4. Has the distinct advantage of developing normalized stress-strain-strength parameters than can be used on subsequent projects involving the same deposit (especially if they have yielded acceptable results based on evaluated field experience).

Unconsolidated-undrained testing:

1. Cannot yield meaningful stress-strain data because such tests do not start from the correct initial stress state and are seriously affected by sample disturbance (even with block quality samples).
2. Often give unreliable and highly scattered $c_u$ data due to varying degrees of sample disturbance plus uncontrolled effects from strain rate and anisotropy.

Discussion topics and research needs:

1. Have researchers conducted programs, such as done for the Champlain Sea clays, on a variety of soil types to well document the specific effects of sample disturbance.
2. Assuming a negative answer, systematic studies comparing results from Recompression and SHANSEP testing on block samples subjected to varying degrees of disturbance are sorely needed for a variety of "ordinary" clays having different amounts of overconsolidation.
3. There is also a need for further research into the causes of sample disturbance. As one example, Baligh (1984) has used the "strain path" technique to evaluate the influence of sampler geometry on strains induced during sampling.

### 2.4. STRENGTH-DEFORMATION CHARACTERISTICS UNDER GENERALIZED STATES OF STRESS

#### 2.4.1. Introduction

A state of stress is defined by the magnitudes and directions of the three principal stresses ($\sigma_1$, $\sigma_2$ and $\sigma_3$) and generalized states of stress denote all possible conditions. For most practical field problems and when comparing various laboratory testing devices, it is usually sufficient to describe basic differences in the applied state of stress (applied stress system) by two variables: (1) the direction of the applied major principal stress relative to the vertical (depositional) direction denoted by the $\delta$ angle; and (2) the relative magnitude of the intermediate principal stress as defined by $b = (\sigma_2 - \sigma_3) / (\sigma_1 - \sigma_3)$. Changes in the values of $\delta$ and b lead to different stress-strain responses due to anisotropy and the $\sigma_2$ effect respectively.
The ability to measure soil behaviour under different applied stress systems, i.e. under generalized states of stress, is required to obtain input parameters for most sophisticated soil models (e.g. certainly for Category I in Table I and often for Category II) and to generate basic knowledge needed to develop improved constitutive relationships. This latter knowledge also helps evaluate likely errors associated with results from tests that do not exactly duplicate in situ stress paths, e.g. use of triaxial test data for a plane strain field problem.
This section focuses on anisotropy because little data and much confusion exist compared to our understanding of $\sigma_2$ effects. After describing different types of anisotropy and illustrating their effects, the section gives an assessment of the current ability (or inability) to measure anisotropy and then concludes with an overview of research needs.

#### 2.4.2. Anisotropy: Initial and Evolving

Consider identical samples from a horizontal deposit wherein the soil was deposited in the vertical (z axis) direction and underwent only one-dimensional strains, i.e. $K_o$ consolidation and rebound. Figure 13(a) shows the sample orientation for testing with the directional shear cell (DSC)

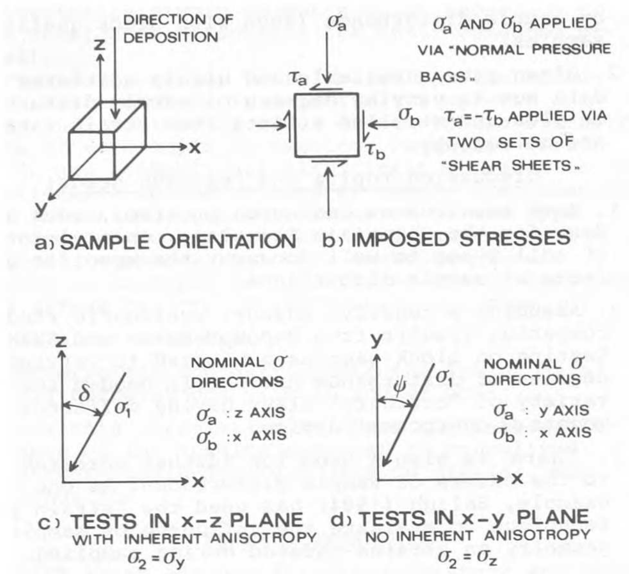



Fig.13: Measurement of Anisotropy with the Directional Shear Cell.

that will be used to illustrate anisotropy. This plane strain shear apparatus, described subsequently, controls the major principal stress direction by varying the normal stresses, $\sigma_a$ and $\sigma_b$, and the shear stresses, $\tau_a = -\tau_b$, acting on four faces of cubical sample constrained between two rigid end plattens, as illustrated in Figure 13(b).

When the cubical sample is placed in the DSC such that shear occurs in the x-z plane [see Figure 13(c)], the device measures the influence of anisotropy by varying the $\sigma_1$ direction between the z-axis and the x-axis, i.e. $\delta$ increasing from 0° to 90°. Such a test series simulates conditions for elements within the failure zone beneath a strip footing. When the cubical sample is placed so that shear occurs in the x-y plane [see Figure 13(d)], the response of the sample is independent of the $\sigma_1$ direction (the $\psi$ angle) since soil properties are the same in all radial directions. This stress system simulates cavity expansion and the self-boring pressuremeter test. The response will be different from the above $\delta = 90°$ test sheared in the x-z plane since the soil is cross-anisotropic. This fact illustrates that $\delta$ and b are not adequate to completely define states of stress since both are plane strain tests having $\delta=90°$, but the first is sheared in the x-z plane and the second in the x-y plane.

Fig.14(a), which plots results from drained tests run in the DSC on loose Leighton Buzzard sand, shows very dramatic effects of anisotropy on stress-strain behaviour. For samples sheared in the x-z plane (varying $\delta$ angle), anisotropy not only greatly reduced the modulus, but also caused a basic change in the shape of the curves. Shearing in the "isotropic" x-y plane produced a response about equal to that for $\delta$ = 45°. Figure 14(b) shows corresponding data from undrained DSC tests performed on resedimented Boston Blue clay having a nominal OCR=4. Although the initial modulus remains constant, anisotropy has a very pronounced effect on the yield stress, the modulus at yield and the peak strength.

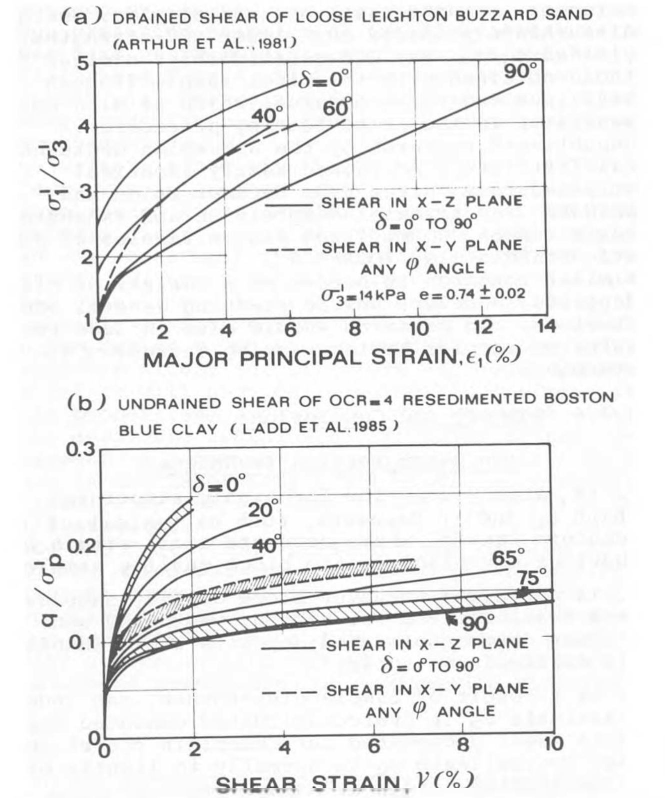



Fig.14: Examples of Inherent Anisotropy for Plane Strain Monotonic Loading in the Directional Shear Cell: (a) Drained Shear of Loose Leighton Buzzard Sand [Arthur et al. (1981)] and (b) Undrained Shear of OCR=4 Resedimented Boston Blue Clay [Ladd et al. (1985)].

The above data were all obtained from tests having an initial isotropic state of stress. Hence the observed anisotropy must have resulted entirely from a preferred soil structure developed during one-dimensional deposition. This type of anisotropy is commonly called inherent anisotropy. Mitchell (1976) summarizes measurements of preferred interparticle contacts in granular materials and of preferred particle orientations (soil fabric) in both sands and clays. Macroscopic variations in fabric can also produce inherent anisotropy; examples are stiff fissured clays and varved glacial lake deposits containing alternating layers of "silt" and "clay" [Ladd et al. (1977)]. In any case, inherent anisotropy will lead to directional changes in basic material properties governing elastic behaviour, plastic behaviour and failure.

Soils can also exhibit directional dependent undrained strengths whenever shearing starts from an anisotropic initial state of stress. Hansen and Gibson (1949) first predicted the existence of this type of anisotropy which, for lack of a better term, will be called initial shear stress anisotropy [Section 2.2.2. of Ladd et al. (1977) used the term stress system induced anisotropy to describe the same phenomenon]. The developed theoretical expressions for the variations in undrained strength with mode of failure for normally consolidated clays as a function of $K_o$, the Hvorslev strength parameters and the equivalent of Skempton's pore pressure parameter $A_f$ [see Duncan and Seed (1966) for an updated version]. Their plane strain predictions showed

| OCR = 1 Clay | $c_u/\sigma'_p$ | |
|---|---|---|
| | Compression ($\delta$=0°) | Extension ($\delta$=90°) |
| Lean Sensitive ($K_o$ = 0.5) | 0.331 | 0.193 |
| Plastic Insensitive ($K_o$ = 0.75) | 0.282 | 0.252 |

where $c_u = q_f = 0.5\ (\sigma_1 - \sigma_3)_f$. It should be emphasized that these strength ratios were computed using isotropic material properties, i.e. assuming no inherent anisotropy. Hence the changes in $c_u$ result solely from the fact that different increments of shear stress are required to produce failure as $\sigma_{1f}$ varies between the vertical ($\delta$ = 0°) and the horizontal ($\delta$=90°) direction.

The above initial shear stress anisotropy helps explain the undrained behaviour of soils and it should be considered when conducting test programs or interpreting $c_u$ test data. But this effect can be theoretically computed from a knowledge of material properties, be they isotropic or anisotropic. Hence from a fundamental viewpoint, it is not a "material" anisotropy.

The anisotropic stress-strain-strength behaviour measured during monotonic loading (shearing) of identical samples will be called initial anisotropy. Soils that have undergone one-dimensional deposition and straining have cross-anisotropic initial material properties due to inherent anisotropy. In addition, when $K_o$ is not equal to unity, undrained behaviour is also affected by initial shear stress anisotropy.

Soils can also attain their initial anisotropy due to prestraining. Results from drained tests run on sand in the directional shear cell will be used to illustrate this strain induced anisotropy. Loose sand was deposited along the z-axis and placed in the DSC for testing in the isotropic x-y plane [Fig.13(d)]. The effects of strain induced anisotropy were then investigated as follows ($\sigma'_3$ always kept constant):

1. Prestrain the initially isotropic test specimen during a first shear to $R = \sigma'_1/\sigma'_3 = 4.0$ with $\sigma_1$ acting at some given direction, say $\psi_A$.

2. Unload the sample and then perform a second shear at a different $\sigma_1$ direction, say $\psi_B$.

3. This produces a rotation in the $\sigma_1$ direction of $\theta = \psi_B - \psi_A$.

Arthur et al. (1977) first conducted such tests and Fig.15 plots results from a similar series run at MIT on the same sand previously used to illustrate inherent anisotropy [shown in Figure 14(a)]. Compared to isotropic sand, the data show that strain induced anisotropy produces: (1) a much stiffer response, with characteristic hyperbolic stress-strain curves, at small rotation angles; and (2) a much softer response, with an almost linear R vs. $\varepsilon_1$ relationship, at large $\theta$ angles. But a fundamental difference exists

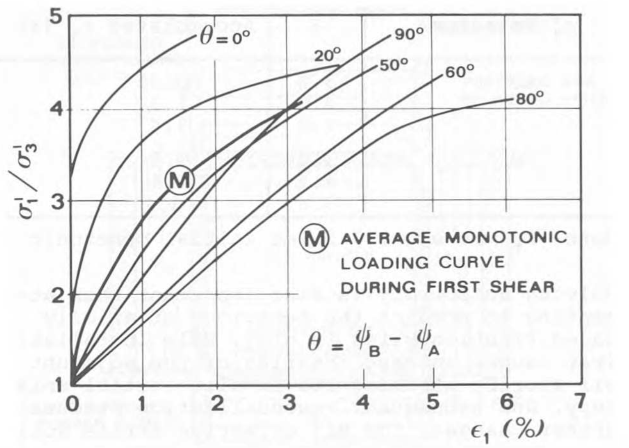



Fig.15: Example of Strain Induced Anisotropy for Loose Leighton Buzzard Sand Tested in the Directional Shear Cell [Arthur et al. (1981)].

between inherent and strain induced anisotropy. Whereas inherent caused a continuous increase in strain as $\delta$ rotated from 0 to 90°, the strains in the tests with strain induced anisotropy rise to a maximum at $\theta$ = 70 ± 10° and then decrease at $\theta$ = 90°. This important difference also occurred with dense sand [Arthur et al. (1977),(1981)].

Initial anisotropy is now generally recognized to be an important aspect of soil behaviour. Advances in soil modeling are such that the theoretical ability to reasonably predict the in situ response of horizontal soil deposits under monotonic loadings having different $\sigma_1$ directions is better than the experimental ability to provide accurate input parameters. Moreover, some soil models have gone one step further by considering evolving anisotropy [Dafalias in RNESE (1983)]. What is meant by evolving anisotropy? It describes how the initial anisotropy of a soil changes due to subsequent stressing or straining. For example, the initial cross-anisotropic properties of a $K_o$ consolidated sample will become less pronounced if unloaded and then subjected to large hydrostatic compression, i.e. the initial anisotropy will evolve towards isotropy. Or if the same sample is stressed at some $\delta \neq 0$ angle to produce substantial plastic strains, unloaded and then sheared at varying $\delta$ angles, the measured response will be different from that of the "virgin" sample. Thus the problem of anisotropy not only entails identification of its initial features but also development of laws describing its evolution. Moreover, both initial and evolving anisotropy can influence all three components of soil modeling: elastic behaviour, plastic behaviour and failure.

Evolving anisotropy is especially important in problems involving large stress reversals; one prime example is the foundation response of a gravity offshore platform subjected to wave action. Arthur et al. (1980) used the directional shear cell to measure the accumulated strains developed during drained continuous cyclic shearing. Typical results for dense Leighton Buzzard sand after 50 cycles wherein $R = \sigma'_1 / \sigma'_3$ was kept approximately constant during each test are:

| $\sigma_1'$ Rotation | R | Accumulated $\varepsilon_1$ (%) |
|---|---|---|
| 40° | 3.3 | 0.25 |
| | 4.0 | 1.3 |
| | 4.8 | 3.4 |
| 70° | 3.4 | 0.8 |
| | 4.2 | 12.4 |
| | 5.6 | ≅ 20 |

(Note: $R_f$ was about 7.5 for initial monotonic loading)

Evolving anisotropy is also important when attempting to predict the behaviour of axially loaded friction piles in clay. Pile installation first causes intense shearing of the adjacent soil thereby altering its in situ initial anisotropy, and subsequent reconsolidation produces further changes. The MIT effective stress soil model developed by Kavvadas and Baligh (1982) considers evolving anisotropy via a kinematic hardening rule that allows continuous rotation of the yield surfaces. Several other soil models [e.g. Prevost and Höeg (1977)] have also incorporated evolving anisotropy in order to predict behaviour for complex stress paths having cyclic loading conditions especially pertinent to offshore construction. However, little experimental data exist that can be used to check the validity of the "laws of evolution" assumed in these models.

### 2.4.3. Experimental Capabilities

This section assesses various laboratory shear devices regarding their capability to measure strength-deformation characteristics under generalized states of stress. It first treats three representative devices considered best suited for basic research into the effects of anisotropy ($\delta$ angle) and the intermediate principal stress (b parameter) on constitutive relationships. It then discusses testing procedures that can be employed in engineering practice to evaluate anisotropy for stability problems and/or to obtain input parameters for anisotropic soil models. The focus is on behaviour during monotonic loading, i.e. initial rather than evolving anisotropy and effects of cyclic loadings.

The writers' specifications for a shear device ideally suited for basic research would include the following characteristics:

1. Have uniform and well-defined states of stress and strain, at least prior to formation of rupture surfaces.
2. Provide accurate data over a wide range of strains and confining stress levels.
3. Be capable of performing both stress and strain controlled loading, the latter usually being essential to measure strain softening behaviour.
4. Be capable of performing both drained and undrained tests on granular and cohesive soils having a well defined stress history and minimal sample disturbance (the effort required to prepare test specimens is often a major expense).
5. Be capable of varying the magnitude and direction of all three principal stresses, or at least be able to:
   a) vary b at constant $\delta$ to measure the effect of $\sigma_2$;
   b) vary $\delta$ at constant b to measure anisotropy.
6. Be relatively easy and inexpensive to operate.

Items 1 and 5a or b are considered essential for any meaningful research and the others are certainly desirable, although specific requirements largely depend upon the research objectives, e.g. "elastic" behaviour versus plastic behaviour and failure conditions.

Before discussing specific devices, some comments on stress-strain uniformity are in order. Most stress-strain data come from devices with rigid end plattens (i.e. the conventional triaxial cell) wherein average strains are computed from boundary displacements and overall volume changes. Stress distributions along the end plattens are seldom measured. Hence little data exist to indicate the possible magnitude of nonuniformities in stress or strain within the samples. In contrast, Arthur and his colleagues at University College London routinely measure strain distributions within samples by using radiography (embedded lead shot) or photography (inked lines on a sample boundary). Their data show substantial variations in strain for presumably very uniform sand within the middle third of triaxial specimens having rigid ends and within flexible boundary shear devices [Arthur et al. (1981)]. This result suggests that more emphasis should be placed on measuring strain distributions in order to check stress-strain uniformity.

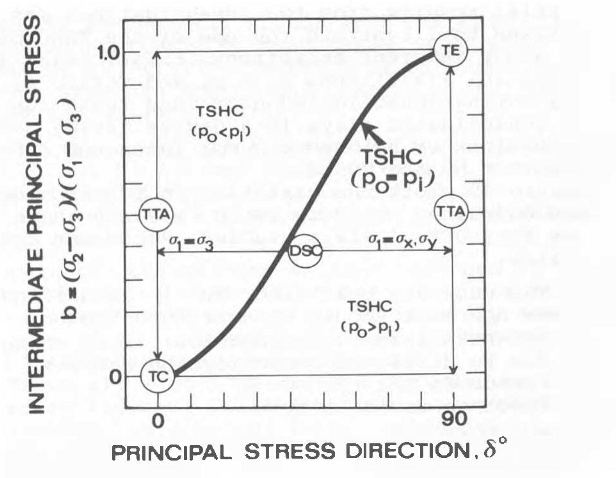
The three shear devices to be discussed are: the true triaxial apparatus (TTA); the directional shear cell (DSC); and the torsional shear hollow cylinder (TSHC). Saada and Townsend (1981) present a comprehensive evaluation of these and other strength testing devices, with emphasis on their research capabilities. They provide extensive references and also a somewhat different viewpoint than expressed here regarding the DSC and TSHC.

Figure 16 illustrates combinations of b and $\delta$ that can be achieved by the three devices and for comparison, the location of conventional triaxial compression and extension tests. Testing of samples trimmed at various inclinations to the vertical depositional direction (z-axis) are excluded because: the specimen cannot be properly $K_o$ consolidated; and bending moments and shearing forces are generated at the ends when an inclined specimen is tested between rigid end plattens [Saada (1970), Saada and Bianchini (1977)].

The true triaxial apparatus (TTA) is ideally suited to investigate $\sigma_2$ effects, and numerous usually cubical devices exist having all rigid, all flexible or mixed boundary conditions. Sture and Desai (1979) and Saada and Townsend (1981) compare their attributes and give references. Most have been used to study sands, with consistent results for $0 < b < 0.5$, but divergent trends at higher b values [e.g. b vs. $\phi'$ in Figure 13 of Ladd et al. (1977)].Whether the differences are real or due to apparatus problems is not clear. Relatively few devices have been developed to test cohesive soils as they are usually more complex, and most data are from remolded materials [e.g. Lade and Musante (1978)].

The TTA has very limited ability to study anisotropy since $\sigma_1$ must act along the x, y or z axes. Hence, for plane strain conditions, it is restricted to simulating:

- $\delta = 0°$, with $\sigma_z > \sigma_x = \sigma_2 > \sigma_y$
- $\delta = 90°$, with $\sigma_z < \sigma_x = \sigma_2 < \sigma_y$
- Cavity extension, $\delta=90°$, with $\sigma_x > \sigma_z = \sigma_2 > \sigma_y$




DSC = DIRECTIONAL SHEAR CELL (SHOWN FOR SHEAR IN x-z PLANE)

TC = TRIAXIAL COMPRESSION

TE = TRIAXIAL EXTENSION

TSHC = TORSIONAL SHEAR HOLLOW CYLINDER (VARYING $\sigma_z, \tau_{z\theta}, p_i, p_o$)

TTA = TRUE TRIAXIAL APPARATUS

Fig.16: Combination of b and $\delta$ which can be Achieved by Various Shear Devices [After Germaine (1982)].

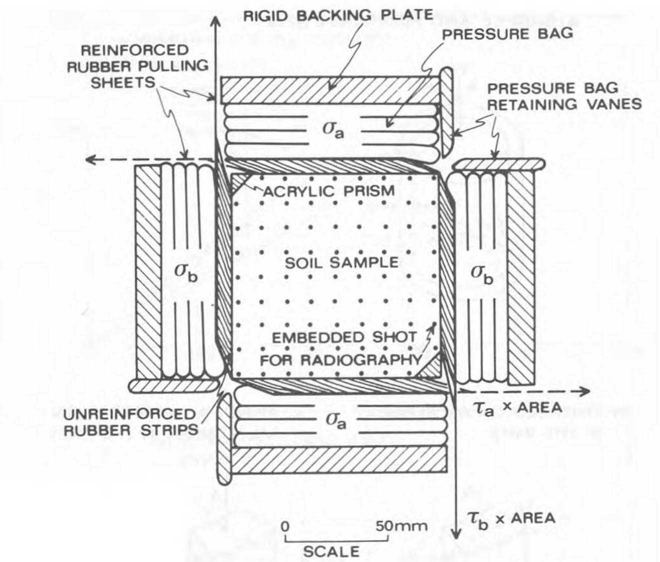



Fig.17: Diagram of Method Used to Apply Normal and Shear Stresses in the Direction Shear Cell [Arthur et al. (1977, 1980 and 1981)].

The directional shear cell (DSC) was specifically developed to investigate anisotropy in sands [Arthur et al. (1977)].

Figure 17 shows a diagram of the method used to apply normal and shear stresses to the four faces of a cubical sample, constrained between two rigid end platens, in order to fully control the $\sigma_1$ direction during plane strain shear. Arthur et al. (1981) describe the operation of its main components: the unique shear sheets for applying the shear stresses $\tau_a$ and $\tau_b$; the pressure bags for applying the normal stresses $\sigma_a$ and $\sigma_b$; and the radiographic-photographic techniques for measuring the distribution of strain magnitudes and directions.

One of the most attractive features of the DSC is the ability to conduct "proof" tests in order to check if the device operates as intended. Samples are sheared in the isotropic x-y plane with different directions of $\sigma_1$ to the y axis, i.e. variations in the $\psi$ angle in Fig.13(d). Such tests require changes in the relative magnitude of the shear stress applied via the two sets of pressure bags, $\sigma_a \neq \sigma_v$, versus that applied via the shear sheets, $\tau_a = -\tau_b$ (see Figs. 13 and 17). If the DSC and the experimental techniques work perfectly, such tests should yield identical stress-strain data for identical samples independent of the $\psi$ angle. Drained tests on sand by Arthur et al. (1977, 1981) and undrained test on overconsolidated clay be Germaine et al. (1985) indicate that the DSC can yield reproducible data having negligible systematic errors.

The DSC is the best plane strain device available for basic research into various types of anisotropy. It also has the added advantage of providing detailed information regarding strain distributions (magnitudes and directions). However, the current University College London-MIT device has certain limitations: (1) stress controlled, hence cannot easily measure strain softening behaviour; (2) restricted to relatively low stress levels (shear stress limited to 50 kPa) and not suited for undrained shear of sands; (3) the radiographic-photographic technique is time consuming and requires a significant capital investment; and (4) reliable data require careful, experienced operators. Sture et al. (1985) describe a more expensive and elaborate DSC device that can operate at higher stress levels and uses surface displacements rather than radiography-photography.

Figure 18 shows the idealized stress conditions that can be achieved in the torsional shear hollow cylinder (TSHC) device. Application of an axial load W, a torque M, and internal and external pressures $p_i$ and $p_o$ generates stresses $\sigma_z$, $\tau_{\theta z}$, $\sigma_r$ and $\sigma_\theta$ in the wall of the specimen. By controlling these stresses, the magnitudes of $\sigma_1$, $\sigma_2$ and $\sigma_3$ can be independently controlled, together with the orientation of the major principal stress direction, $\delta$ angle. Tests performed with identical inner and outer cell pressures require that $\sigma_2 = \sigma_r = p_i = p_o$. Consequently, changes in the $\delta$ angle must be accompanied by changes in the intermediate principal stress condition as given by the relationship $b = \sin^2 \delta$, which is plotted in Fig.16. For example, for tests run at constant $\sigma_2 = p_i = p_o$:

- Increasing $\sigma_z$ gives $\delta = 0°$ and $b = 0$ (triaxial compression)
- Increasing $\tau_{\theta z}$ gives $\delta = 45°$ and $b = 0.5$
- Decreasing $\sigma_z$ gives $\delta = 90°$ and $b = 1.0$ (triaxial extension).

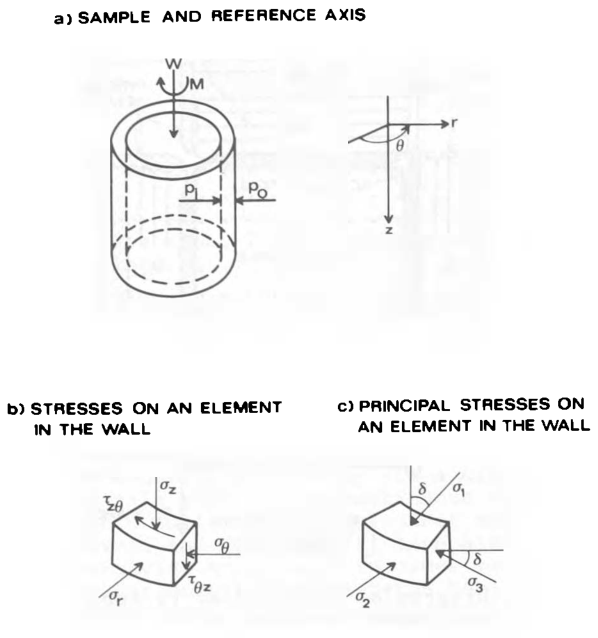



Fig.18: Idealized Stress Conditions in the Torsional Shear Hollow Cylinder Device [From Hight et al. (1983)]

But TSHC tests run with unequal inner and outer cell pressures can, in theory, obtain any combination of δ and b. In other words, the device can study the influence of changing b at constant δ and also the effect of changing δ at constant b. For example, tests to measure inherent anisotropy at b = 0.5 by maintaining $\sigma_2 = \sigma_r = 0.5\ (\sigma_z + \sigma_\theta)$ could be performed as follows:

- $p_o \leq p_i$ and $\sigma_z \geq \sigma_r$ lead to δ increasing from 0° to 45° by increasing the applied torque M, i.e. increasing $\tau_{\theta z}$.
- $p_o \geq p_i$ and $\sigma_z \leq \sigma_r$ lead to δ increasing from 45° to 90° by decreasing the applied torque M, i.e. decreasing $\tau_{\theta z}$.

However, in reality, the TSHC device cannot be used for testing at all combinations of b and δ due to the nonuniformity of stress (and hence strain) across the specimen wall which arises because of the specimen's curvature, and restraint at the end platens. Saada and Townsend (1981) discuss the influence of device geometry on nonuniformities for testing with $p_o = p_i$. Hight et al. (1983) also analyze the added complications arising from having unequal inner and outer pressures and summarize the sample dimensions, loading capabilities and research objectives of various hollow cylinder apparatuses. Both references indicate that nonuniformities caused serious problems for many of the devices that have been used to investigate b and/or δ. It should be noted that stress-strain variations occur across the wall even when boundary stresses are uniformly applied, even if there is no end restraint due to either the application of torque or different inner and outer pressures.

Partial results from two investigations are summarized to illustrate the use of the TSHC device to study inherent anisotropy. First, Saada and Bianchini (1975) kept $p_o = p_i$ and varied $\sigma_z$ and $\tau_{\theta z}$ during consolidated-undrained tests run on $K_o$ consolidated clays in a device having nominal dimensions of height = 150 mm; inner and outer diameters = 50 and 70 mm.
Figure 19 plots the variation in friction angle and undrained strength ratio versus δ angle of the four OCR = 1 clays studied. Pertinent comments follow.

1. The data are scattered, but it is difficult to know how much should be attributed to:
   - Non-normalized clay behaviour, i.e. changes due to different consolidation stresses (some trends do exist).
   - Specimen variability.
2. The friction angle tends to increase and then remain constant at δ > 45°. Saada and Bianchini (1975) considered ϕ' to "vary in a completely random fashion between 26.7° and 54.9°" and attributed the large variation to the Mohr-Coulomb failure criterion not being valid for anisotropic materials. Although inherent anisotropy can and does affect ϕ', the writers believe that changes in the intermediate principal stress condition should also be considered since $b = \sin^2 \delta$ for this test program. The systematic increase in b could

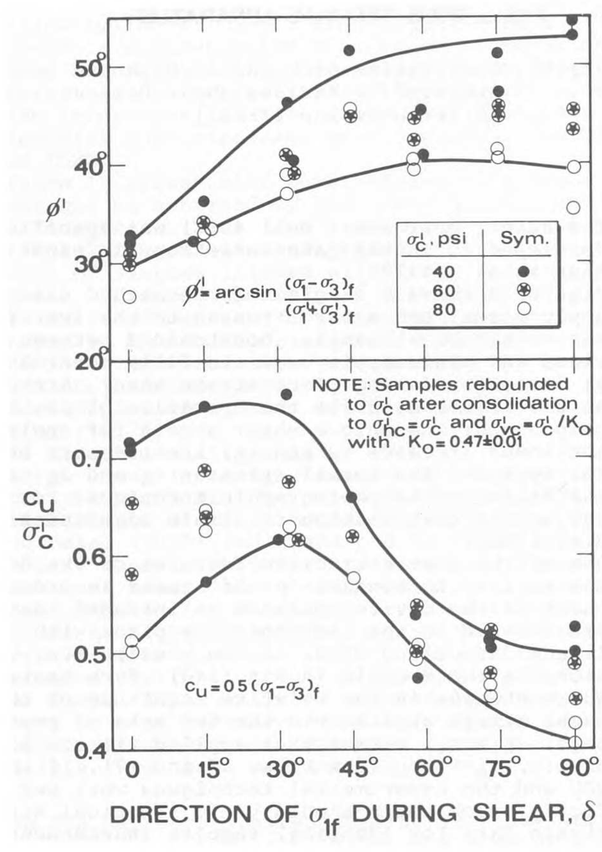



Fig.19: Results of CIU Torsional Shear Hollow Cylinder Tests on Edgar Plastic Kaolin [Data Plotted from Saada and Bianchini (1975)].

explain the general trends in $\phi'$ if this clay behaved similarly to many sands, e.g. Fig.13 of Ladd et al. (1977). (Note: b vs. $\phi'$ data for clays are both scarce and contradictory; see Lade and Musante (1978).

3. The values of $c_u/\sigma_c'$ generally increase between $\delta = 0°$ and $\delta = 30°$ and then decrease at greater inclinations of $\sigma_{1f}$. But, as with $\phi'$ above, one cannot separate the relative effects of changes in $\delta$ versus b.

In summary, the writers believe that use of the TSHC with equal inner and outer pressures for basic research into anisotropy is severely hampered because of the unknown influence of changes in the intermediate stress condition.

Hight et al. (1983) describe a TSHC device developed at Imperial College starting in 1975 for investigating anisotropy. Rather extensive theoretical analyses of stress-strain nonuniformities, especially with $p_o \neq p_i$, lead to an apparatus having the following features:

1. Sample dimensions: height = 254 mm; inner and outer diameters = 203 and 254 mm.
2. Stress controlled shearing with inside - the - cell measurements of axial load and torque and, for the central portion of the sample, inside - cell measurements of radial, vertical and circumferential displacements.
3. Ability to conduct either consolidated-drained or undrained tests on saturated sands deposited one dimensionally.
4. Ability to vary b at constant $\delta$ or to vary $\delta$ at constant b (important for measuring anisotropy), but wihin certain limits since $p_o/p_i$ is restricted to between 0.9 and 1.2 to minimize nonuniformities in strain and particularly stress (pre-failure stress states are considered "reasonably free from error", Symes et al., 1984). This restriction means that tests cannot be run with combinations of b and $\delta$ that fall in the upper left and lower right portions of Fig.16.

Symes et al. (1984) used this device to measure the inherent anisotropy exhibited by medium-loose Ham river sand during undrained shear. The sand was deposited through water to form the hollow cylindrical specimen, isotropically consolidated to $\sigma_c' = 200$ kPa with backpressure (400 kPa), "presheared" via drained triaxial compression to $\sigma_1'/\sigma_3' = 2$ to check repeatability (considered to have caused negligible strain induced anisotropy) and then shear undrained at b = 0.50 while keeping $\sigma_o$ constant at 600 kPA. Figure 20 plots normalized data from three CIU tests sheared at $\delta = 0°$, 24.5° and 45°. These results show that inherent anisotropy caused:

1. A dramatic change in the "small strain" peak undrained strength, i.e. $c_u/\sigma_c'$ decreasing from 0.44 to 0.23, due to a combination of higher pore pressures and lower obliquities with increasing $\delta$ angle.
2. A fairly significant decrease in the "large strain" failure envelope.

These authors also present data wherein samples were first sheared undrained at $\delta = 0°$ and subsequently at $\delta = 45°$, and vice versa, that produced results considered reasonable by the writers. Hence the device appears to have the important capability of investigating both initial and evolving anisotropy.

Based on the above results, the writers conclude that the Imperial College TSHC device offers extremely attractive research capabilities regarding detailed studies of anisotropy under both drained and undrained shear conditions. However, this exciting and greatly needed new testing ability is very costly, both to construct and to operate, and also restricted to pre-failure stress states. For these reasons, the directional shear cell is perhaps more "cost effective" for studying anisotropy, especially at large strains, but only under plane strain conditions and for low strength materials.

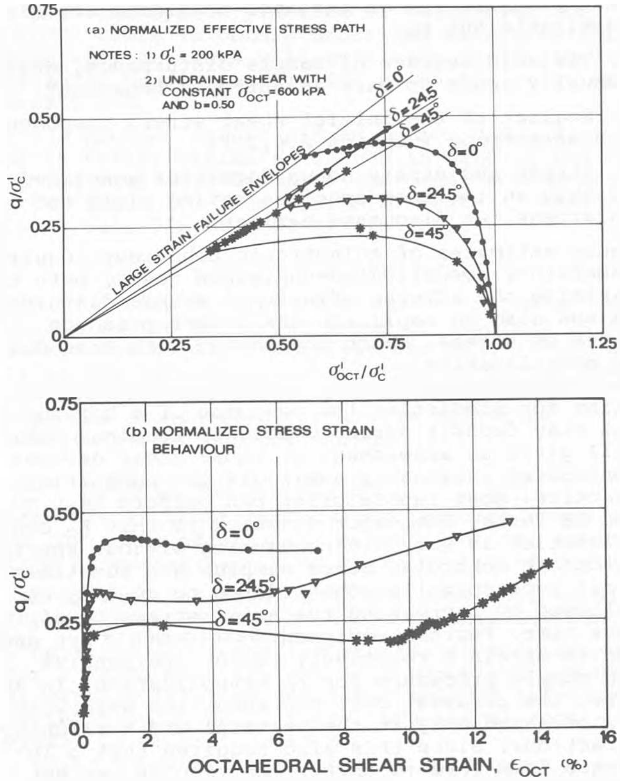



Fig.20: Results of CIU Torsional Shear Hollow Cylinder Tests on Medium-Loose Ham River Sand; (a) Normalized Effective Stress Paths and (b) Normalized Stress-Strain Behaviour [From Symes et al. (1984)].

## Evaluating Clay Anisotropy in Engineering Practice

The directional shear cell and torsional shear hollow cylinder test are research devices ill-suited for engineering practice. On the other hand, major projects often require input parameters for anisotropic soil models needed for finite element analyses. Also most stability evaluations should consider undrained anisotropy when selecting design strengths, at least for cases involving stage construction, unusual loading conditions (e.g. ice forces on Arctic gravity structures) and similar situations where empirical approaches have obvious deficiencies. The following gives the writers' views on what test devices and procedures should or should not be used.

In situ testing cannot give input parameters for soil models nor a reliable estimate of the in situ undrained strength anisotropy. Ladd et al. (1977) discussed specific limitations of the field vane and self-boring pressuremeter tests for such purposes. Laboratory unconsolidated-undrained (UU) test programs run on inclined specimens are also unreliable due to:

1. Variable degrees of sample disturbance, which usually tends to mask anisotropic behaviour.
2. Neglect of the initial shear stress component of anisotropy (Section 2.4.2.).
3. Stress and strain nonuniformities generated within an inclined specimen having rigid end plattens, as discussed earlier.

Hence estimates of anisotropic behaviour require laboratory consolidated-undrained tests, both to minimize the adverse effects of sample disturbance and also to replicate the proper preshear state of stress, which may change with time due to consolidation.

The discussion is mainly restricted to $CK_oU$ testing for predicting the response of a horizontal clay deposit during monotonic loading. Table VIII gives an assessment of those shear devices considered reasonably available for consulting practice. Most laboratories can perform $CK_oU$ TC and TE tests. The major drawback is that $K_o$ consolidation is very time consuming without special automatic controls. Hence samples are sometimes first isotropically consolidated to $\sigma'_{hc} = K_o\ \sigma'_{vc}$, followed by increasing the axial stress to $\sigma'_{vc}$ to save time. Further equipment developments are needed to obtain a relatively rapid, inexpensive and simple procedure for $K_o$ consolidation. In any case, the triaxial cell can shear the soil with $\sigma_{1f}$ oriented only in the vertical and horizontal directions. Since this also requires that b increase from zero to unity, the results reflect the combined influence of anisotropy and differences in the $\sigma_2$ condition. In this regard, PSC and PSE tests offer an advantage by maintaining plane strain conditions. But such devices are less available and generally more costly to operate.

When interpreting TC/TE data, one can:

1. Simply ignore the change in the b condition, since this should yield conservative results (see below).
2. Employ an anisotropic soil model that uses TC/TE data as input parameters [e.g. Prevost and Höeg (1977), Prevost (1978, 1979)] to predict plane strain behaviour.
3. Use past experience to correct the strengths, e.g. the following information from Ladd et al. (1977) for normally consolidated clays.

| Loading Direction | $\frac{c_u\text{(Triaxial)}}{c_u\text{(Plane Strain)}}$ | |
|---|---|---|
| Compression ($\delta = 0°$) | 0.92 ± 0.05 | (6 clays) |
| Extension ($\delta = 90°$) | 0.82 ± 0.02 | (4 clays) |

The amount of the continuous rotation of the $\sigma_1$ direction which occurs in the Geonor direct simple shear (DSS) device is of unknown magnitude since the state of stress cannot be defined from a knowledge of the stresses on only one plane, i.e. $\tau_h$ and $\sigma'_v$. Moreover, the basic configuration of the DSS causes nonuniform stress-strain conditions within the sample. Theoretical analyses have led some researchers to conclude that these nonuniformities preclude "reliable stress-strain relations or absolute failure values" [Saada and Townsend (1981)]. However, the writers agree with Vucetic and Lacasse (1982) who believe that such a view is too pessimistic since yielding of the clay will reduce the stress concentrations. Those authors also present experimental DSS data showing little effect of height to diameter ratio which supports that conclusion. Nevertheless, there is

TABLE VIII

Assessment of Laboratory Shear Devices Available for Measuring Anisotropic Stress-Strain-Strength Parameters of Clays in Engineering Practice (Mainly $CK_oU$ Testing)

| Apparatus | b & δ Capabilities | Reliable Stress-Strain | Reliable c' - φ' | Remarks & References to Typical Equipment |
|---|---|---|---|---|
| 1. Triaxial Cell (TC & TE) | δ=0° at b=0 and δ=90° at b=1 | Yes | Yes | . $K_o$ consolidation difficult without special controls [Campanella and Vaid (1972)]<br>. Controlled stress-path testing very advantageous, but expensive [Bishop and Wesley (1975)] |
| 2. Plain Strain (PSC & PSE) | δ=0° & 90° with plane strain b | Yes | Yes | . More complex and costly to operate than triaxial<br>. Vaid and Campanella (1974) |
| 3. Geonor Direct Simple Shear (DSS) | unknown δ=45° ± 15° with plane strain b | Yes(?) | No | . Unknown state of stress<br>. Least expensive $CK_oU$ test with rapid $K_o$ consolidation<br>. Can vary $\tau_{hc}/\sigma'_{vc}$<br>. Bjerrum and Landva (1966) |
| 4. Torsional Shear Hollow Cylinder with $p_i = p_o$ (TSHC) | δ=0° to 90° with b=$\sin^2\delta$ | Yes | Yes | . Available with controlled stress-path testing [Saada and Townsend (1981)]<br>. Used extensively for studying anisotropy of clays by Saada et al. but not starting from $K_o$ stresses |

no generally valid method to determine the values of $0.5(\sigma_1 - \sigma_3)_f$ and $\delta$ in DSS tests. So why perform DSS tests?
The Norwegian Geotechnical Institute (NGI) and MIT have used the DSS test to simulate the horizontal portion of a failure surface as part of a "standard" procedure for considering anisotropy, either via the "ADP" Recompression technique [Bjerrum (1973)] or by SHANSEP [Ladd and Foott (1974)]. The writers know of several major geotechnical firms, oil companies, etc. who also use the DSS test in a similar manner.
The DSS device was used by Soydemir (1976) to investigate stress-strain behaviour at various inclinations of the failure plane via consolidation with different $\tau_{hc}/\sigma'_{vc}$ ratios and then shearing with $\tau_h$ acting in the same or opposite direction to $\tau_{hc}$. The writers' judge such use as reasonable for engineering practice, but it offers little advantage over more conventional $CK_oU$ TC, DSS, TE test programs.
Finally, Ladd (1981) considers that the $c_u=\tau_h(max)$ measured in $CK_oU$ DSS tests gives a reasonable estimate of the average $c_u$ appropriate for evaluating foundation stability. Such tests are also much simpler and less expensive than $CK_oU$ TC/TE testing.
The torsional shear hollow cylinder device used with equal inner and outer pressures enables one to test along the S-shaped combination of b and $\delta$ plotted in Fig.16. As such, it should provide more extensive information than can be obtained from a combination of $CK_oU$ TC, DSS and TE tests, especially given the uncertain interpretation of DSS data. Moreover, Saada and his co-workers at Case Institute of Technology have used their fully automated device to conduct hundreds of tests on $K_o$ consolidated clays, mostly with remolded specimens but also with undisturbed samples. However, the writers withhold judgement on its use in practice since: (1) the scatter shown in published data is larger than one would like to see; (2) they have not seen data wherein shear started from $K_o$ stresses for comparison with results from other devices.
Figure 21 summarizes values of $c_u/\sigma'_{vc}$ measured in $CK_oU$ TC, DSS and TE tests run on a variety of normally consolidated clays. It shows that:

1. Less plastic, and often more sensitive, clays tend to have higher anisotropy than more plastic clays.

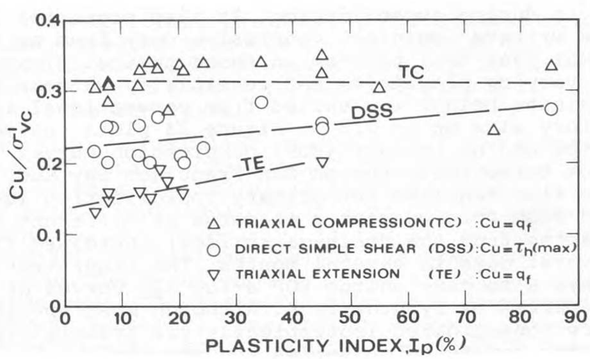



Fig.21: Undrained Strength Anisotropy from $CK_oU$ Tests Performed on Normally Consolidated Clays [Data from Lefebvre et al. (1983), Vaid and Campanella (1974) and Various Publications by MIT and NGI].

2. The wide spread practice of using CIUC (or $CK_oUC$) tests to obtain undrained strengths for stability analyses will generally yield significantly unsafe results for clays of low to moderate OCR.

If the project cannot afford to run DSS and/or TC and TE tests, estimates of the in situ $c_u$ for evaluating stability during stage construction are better predicted from oedometer tests to measure the in situ OCR and the relationship

$$c_u/\sigma'_{vc} = (0.23 \pm 0.04)\ (OCR)^{0.8}$$

than from CIUC testing. With major projects, $CK_oU$ testing having different modes of shearing is considered essential. Koutsoftas and Ladd (1984) describe the "strain compatibility" technique developed at MIT to evaluate the influence of anisotropy and progressive failure in selecting strength parameters for design.

2.4.4. Summary and Conclusions

There are at least two obvious research needs on the influence of anisotropy and $\sigma_2$ on the generalized stress-strain-strength behaviour of soil:

1. Comprehensive studies of the same soil in different shear devices to evaluate this influence.
2. Detailed and intensive studies of anisotropy for a range of a typical soil types e.g., sands at varying relative densities and OCR and clays having varying sensitivity, OCR and $\sigma'_p$ mechanisms, in order to obtain:
   - basic information on initial anisotropy
   - data that can help to develop rules controlling evolving anisotropy

   Until such information becomes available it is not possible to evaluate the errors involved in using "conventional" shear devices such as the TX, PS and DSS for assessing the stress-strain-strength relationships of different natural soils.

## 2.5. TIME EFFECTS

### 2.5.1. Introduction

Time effects in this section refer to: (1) the influence of strain rate on properties measured during consolidation and strength testing; (2) creep, which reflects continued straining at constant applied stress; and (3) relaxation, which refers to changes in stress at constant applied strain. The physical phenomena that cause time effects remain poorly understood and relatively little data exist to show the interrelationship among strain rate, creep and relaxation, except perhaps for one-dimensional consolidation testing. Hence it is not surprising that attempts to develop constitutive relationships to model all aspects of time dependent stress-strain behaviour have met with limited success.
The influence of time effects on various types of conventional laboratory tests can be divided into the following categories:

1. Time effects observed prior to testing, e.g. variations in sample storage time. Assuming that storage occurs at constant composition and volume, then any change in properties presumably results from thixotropy [see Mitchell (1976), Chapter II].
2. Time effects observed during and/or after the end of primary consolidation. This includes various types of consolidation tests (incremental

oedometer, constant rate of strain, etc.) and the consolidation phase prior to CU or CD shear tests. Secondary compression (also called "aging") refers to the specific case of drained creep that follows primary consolidation as typically measured during one-dimensional consolidation testing. Whether the same physical phenomena that lead to secondary compression also have a significant effect on what happens during primary consolidation has been a highly controversial topic [e.g. Ladd et al. (1977)].

3. Time effects observed during drained and undrained shear testing. These usually involve either the influence of strain rate on measured behaviour or creep tests which measure strain versus time at constant applied stress.

The writers had originally intended to discuss several aspects of the above topics, but encountered another time effect, namely the lack thereof. Hence this section is restricted to a treatment of time effects during consolidation, where recent research has clarified some important controversial issues. It will specifically address three items:

1. Effect of drainage height on the location of the end-of-primary (EOP) compression curve and on the applicability of Terzaghi's theory to predict field rates of consolidation.
2. Laboratory measurements of the preconsolidation pressure, $\sigma'_p$.
3. Possible changes in $K_o$ during secondary compression.

### 2.5.2. Time Effects During One-Dimensional Consolidation

The most important question is whether or not creep acts as a separate phenomenon while excess pore pressures dissipate during primary consolidation. Take two extreme cases, Hypotheses A and B. Hypothesis A assumes that creep occurs only after the end of primary consolidation (i.e. after dissipation of excess pore pressures), with secondary compression being caused by the same physical mechanisms responsible for volume changes during an increase in effective stress (e.g. deformation, slippage and reorientation of particles; changes in double layer thickness; distortion of "adsorbed" water films; etc.) [e.g. Ladd (1973), Mesri and Godlewski (1977)]. As illustrated in Fig.22, Hypothesis A predicts that sample thickness, i.e. drainage height ($H_d$) and hence the time required for pore pressure dissipation, has no effect on the location of the end of primary (EOP) compression curve and hence on the value of $\sigma'_p$. Furthermore, the strain versus log t relationship for a given load increment is simply displaced in proportion to $H_d^2$, as commonly assumed in practice.

Hypothesis B assumes that some sort of "structural viscosity" is responsible for creep [e.g. due to the viscosity of adsorbed water films à la Bjerrum (1973)], that this phenomenon occurs during pore pressure dissipation, and therefore that the strain at the end of primary consolidation increases. The precise effects depend upon the specific rheologic model and input parameters. Those shown in Fig.22 are representative of predictions by Garlanger (1972) based on the soil model described by Bjerrum (1967). It predicts a significant shift in the location of the field versus laboratory EOP compression curves, with a corresponding decrease in $\sigma'_p$ and increased consolidation settlements.

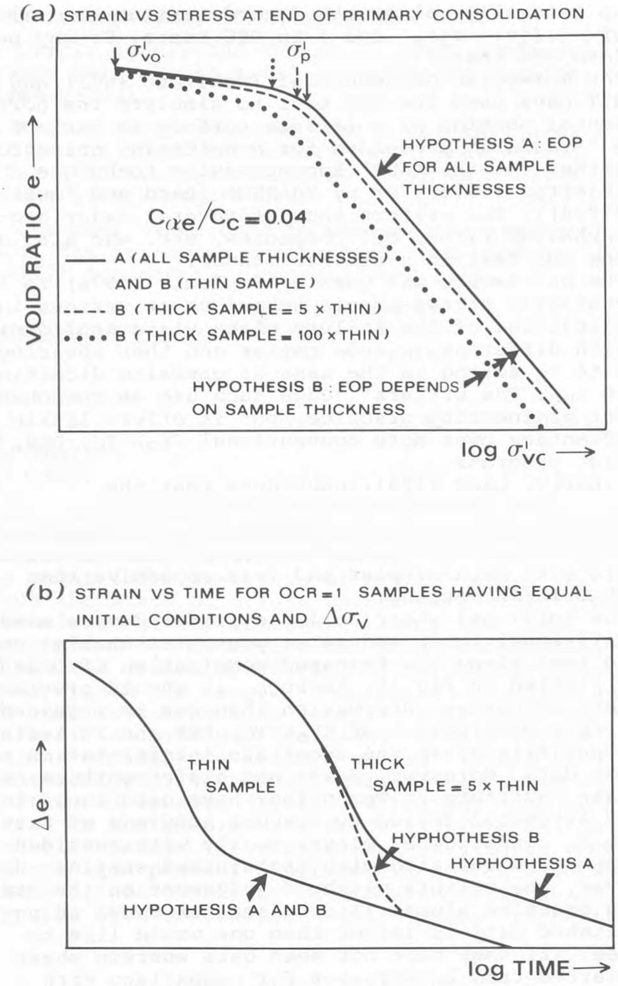



Fig.22: Illustration of Hypotheses A and B in Terms of (a) Strain vs. Stress, and (b) Strain vs. Time.

Mesri and Choi's (1985) paper to this conference reviews past attempts to investigate creep effects during consolidation. It also presents, in the writers' opinion, conclusive data from an eight year test program on three natural clays of varying plasticity and sensitivity wherein the drainage height was varied from conventional laboratory size up to 0.5 m. Figure 23 plots the measured end of primary (EOP) compression curves from three tests run on San Francisco Bay Mud. The time required for primary consolidation (defined from pore pressures measured at different distances from the drainage surface) increased from several days to several months. The other two clays also gave unique EOP e-log $\sigma'_{vc}$ curves as predicted by Hypothesis A. Although most specimens were consolidated isotropically via triaxial specimens connected in series the results should also apply to $K_o$ consolidation since both conditions should have the same $C_{\alpha e}/C_c$ ratio [Mesri and Choi (1984)], where $C_{\alpha e}$ = de / dlogt and $C_c$ = de/log $\sigma'_{vc}$. Mesri and Choi (1985) conclude that their results and "the existing reliable data in the literature support the concept of a unique EOP e-log $\sigma'_{vc}$ cur

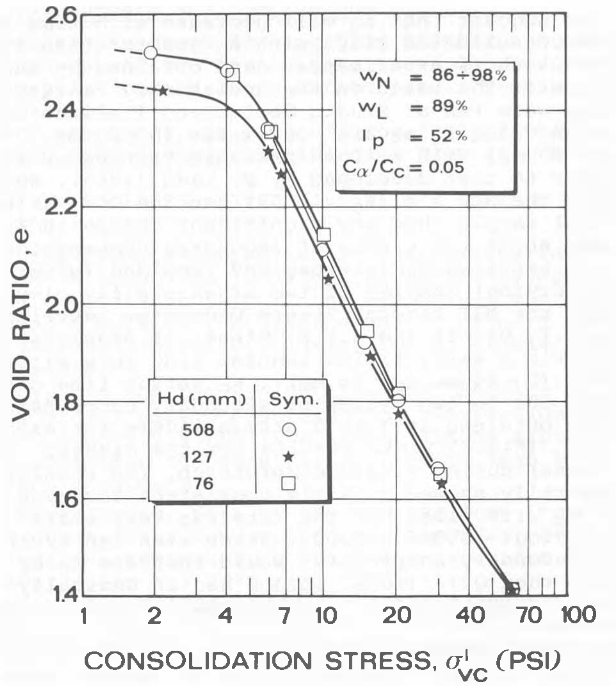



Fig.23: End of Primary Compression Curves for San Francisco Bay Mud with Varying Drainage Heights [from Mesri and Choi, (1985)].

ve for any soft clay". The writers agree and conclude that laboratory EOP consolidation test data can be used to predict the field preconsolidation pressure and the amount of consolidation settlement (for $K_o$ conditions and neglecting sample disturbance).
The specific problem of how time effects influence the $\sigma'_p$ measured in laboratory one-dimensional consolidation tests will now be addressed. First consider standard incremental oedometer tests, where it is well established that:

1. Since $C_{\alpha e}/C_c$ for a given clay is essentially constant during both recompression and virgin compression [e.g. Mesri and Godlewski (1977), Mesri and Choi (1985)].
2. Then the compression curve for consolidation times greater than the end of primary ($t_c > t_p$) will be displaced downward [similar to Curve A versus Curve B in Fig.22(a)] and give a $\sigma'_p$ less than the EOP curve.

Similarly, controlled gradient (CG) and constant rate of strain (CRS) consolidation tests will underestimate the EOP $\sigma'_p$ if the rate is so slow as to permit additional straining due to secondary compression. Conversely, if the EOP curve is independent of drainage height, as previously concluded, the $d\varepsilon/dt$ should not affect $\sigma'_p$ at strain rates equal to or greater than that required to prevent secondary compression. However, extensive data exist from CG and CRS tests which contradict that conclusion. For example, the Leroueil et al. (1983) summary of results for 11 clays typically shows a 10% increase in $\sigma'_p$ per log cycle increase in rate of strain. The writers do not know why this occurs at the higher strain rates, but do offer two possible explanations [these being in agreement with Mesri (1984)]:

1. CG and CRS tests run at "fast" rates have significant pore pressure (effective stress) gradients across the specimen such that resultant variations in compressibility and permeability may invalidate Terzaghi's theory used to interpret the results.
2. At strain rates faster than occur near the end of primary consolidation in conventional incremental tests, many clays may indeed exhibit a "structural viscosity".

Mesri (1984) gives the following relationship to select the strain rate to be used in CRS tests in order to obtain the same $\sigma'_p$ as obtained from EOP incremental oedometer compression curves:

$$\frac{d\varepsilon}{dt} = \frac{K_o}{2\,C_c/C_k\ H_d^2}\ \frac{\sigma'_p}{\gamma_w}\ \frac{C_{\alpha e}}{C_c}$$

where:
$C_c$ = $-de/d\log\sigma'_{vc}$
$K_o$ = initial coefficient of permeability at the in situ void ratio
$\gamma_w$ = unit weight of water
$H_d$ = drainage height
$C_{\alpha e}$ = $-de/d\log t$
$C_K$ = $de/d\log k$

Two empirical correlations are helpful when using this relationship. Mesri and Choi (1985) report $C_{\alpha e}/C_c$ equal to 0.04 ± 0.01 for "a majority of inorganic soft clays" and equal to 0.05 ± 0.01 for "highly organic plastic clays". For the permeability change index, Tavenas et al. (1983) conclude that $C_k = 0.5\ e_o$ is a reasonable practical approximation for clays having an initial void ratio between 0.8 and 3.0. They also show that $C_c/C_k$ varies over a wide range and cannot be easily correlated (at least for highly structured Canadian clays).

## Non-Linear Consolidation

The conventional Terzaghi formulation assumes a constant compressibility during consolidation so that the rate of pore pressure dissipation, du/dt, at any depth should be directly proportional to changes in void ratio. Hence field cases wherein settlements continue while the excess pore pressures remain practically constant have been attributed to abnormal behaviour, with creep often being cited as the possible cause. Mesri (1979) and Mesri and Choi (1979) analyzed one such case, the Gloucester Test Fill reported by Crawford and Burn (1976). They reformulated the Terzaghi theory to account for the highly non-linear void ratio vs. effective stress relationship for this overconsolidated clay having an extremely high compressibility just beyond $\sigma'_p$, see Fig.24(a). They also used a linear e vs. log permeability relationship controlled by the value of $C_k$. The results, plotted in Fig.24(b), show:

1. Rapid pore pressure dissipation with little settlement at early times, as should be expected during consolidation along the flat recompression curve.
2. Then when $\sigma'_{vc}$ exceeds $\sigma'_{vp}$, there is substantial settlement with relatively little pore pressure dissipation because of the very steep e-log $\sigma'_{vc}$ curve during "collapse" of the clay structure.

Hence, as emphasized by Mesri (1979) and Mesri and Choi (1979), the abnormal pore pressure behaviour observed in highly structured clay deposits can be readily explained by nonlinear compressibi-

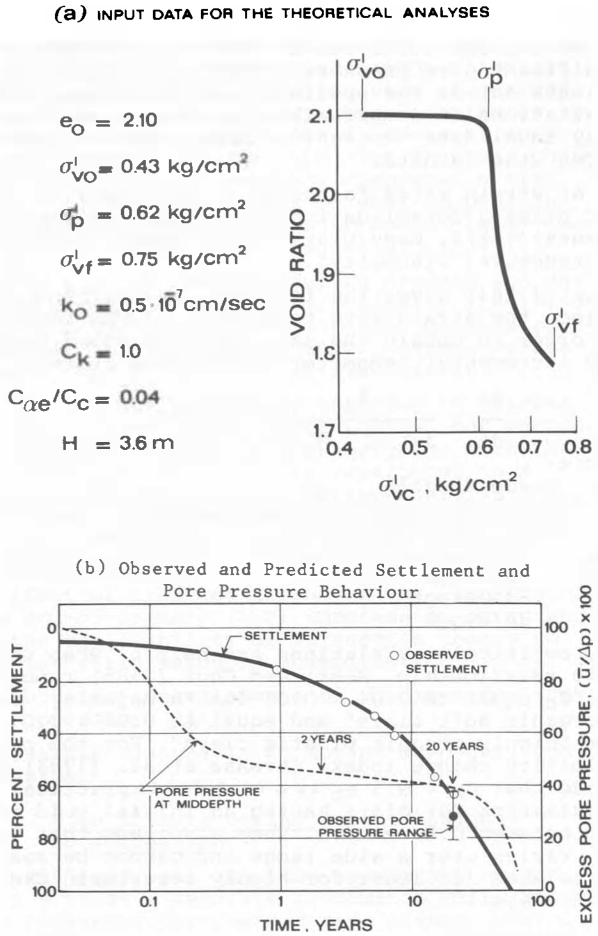



Fig.24: Consolidation Analysis of the Gloucester, Canada, Test Fill [Mesri (1979)].

lity rather than some sort of creep phenomenon.

<u>Changes in $K_o$ During Secondary Compression</u>

Schmertmann (1983) showed that no consensus exists within the geotechnical profession regarding the question: does $K_o$ of a normally consolidated cohesive soil increase, stay the same, or decrease during secondary compression (aging)? The answer has practical implications when attempting to estimate the in situ $\sigma'_{ho}$ in deposits which have developed their $\sigma'_p$ due to secondary compression. In these cases, the laboratory $K_o$ vs. OCR relationship obtained via mechanical overconsolidation will obviously overestimate $K_o$ for "aged" deposits if $K_o$ remains constant or decreases during secondary compression. Kavazanjian and Mitchell (1984) conclude that $K_o$ should increase with time for low OCR clays, "approaching a value of 1.0 over geologic time", based on laboratory triaxial cell data for two clays (undisturbed San Francisco Bay Mud and a compacted kaolinite) and from a theoretical analysis based on the Singh-Mitchell three-parameter creep equation [Mitchell (1976)] and assuming a constant rate of secondary compression, $C_\alpha$. They also suggest that $K_o$ will decrease with time for overconsolidated clays with $K_o$ greater than 1.0. Fig.25 shows experimental data obtained by the writers. The tests on the undisturbed Panigaglia clay were run at Studio Geotecnico Italiano of Milan using a "square" oedometer (D = 50 mm, $H_o$ = 20 mm) with a flush pressure transducer similar to that developed by R. Ladd (1965). Neither the OCR = 1 ($K_o \cong 0.59$) nor the OCR = 10 ($K_o$= =1.6) sample show any significant change in $K_o$ over about two cycles of secondary compression. Four tests on undisturbed and remolded (after air drying) samples of two organic silty clays used the MIT Lateral Stress Oedometer [developed by R.T. Martin and A.E.Z. Wissa, it measures $\sigma_{ho}$ via a water filled annular ring in a circular cell (D = 64 mm, $H_o$ = 28 mm)]. $K_o$ versus time data over one to two cycles of secondary compression were obtained at 3 to 5 stress levels for each test (Fig.25 plots results for the highest stress) during virgin compression. The results generally showed a fairly consistent increase in $K_o$ with time, but the rate was very small, $\Delta K_o/\Delta \log t = 0.007 \pm 0.002$. Hence even ten cycles of secondary compression would increase $K_o$ by less than 0.1. (Note: Soil A had an unusually high friction angle, which probably explains its low $K_o$).

The above results were obtained on soils having values of $C_\alpha/C_c$ representative of typical clays. Consequently, the writers believe that the views expressed by Kavazanjian and Mitchell (1984) either do not apply to all cohesive soils or are premature regarding their significance over geologic log time cycles of practical concern.

| PANIGAGLIA CLAY | | SOIL A | SOIL B |
|---|---|---|---|
| $W_L$ = 65% , $I_p$ = 40% , $C_\alpha/C_{CR}$ = 0.06 ± 0.01 | | □ UNDISTURBED $W_L$ = 138 %, $I_p$ = 78 % $\sigma'_{vc}$ = 60 kPa | Δ UNDISTURBED $W_L$ = 66 %, $I_p$ = 32 % $\sigma'_{vc}$ = 390 kPa |
| O AT OCR = 1 $\sigma'_{vc}$ = 1000 kPa T = 19.8 ± 0.5°C | ● AT OCR = 10 $\sigma'_{vc}$ = 475 kPa T = 21.5 ± 0.8°C | ◇ REMOLDED $W_L$ = 84 %, $I_p$ = 30% $\sigma'_{vc}$ = 245 kPa | ∇ REMOLDED $W_L$ = 50 %, $I_p$ = 17 % $\sigma'_{vc}$ = 390 kPa |

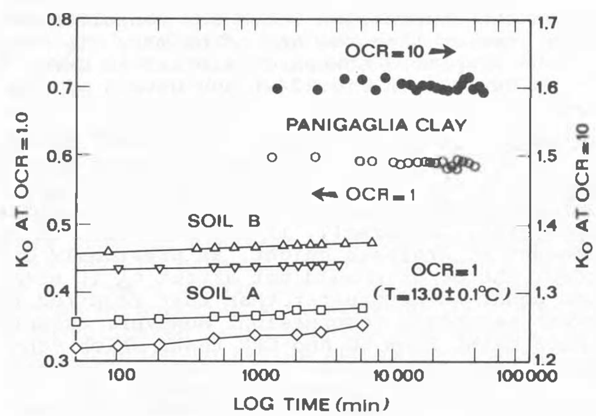



Fig.25: Coefficient of Earth Pressure at Rest vs. time for Undisturbed and Remoulded Clay.

### 2.5.3. Summary and Conclusions

Physical phenomena causing time effects are poorly understood and little data exist to show the interrelationship among strain rate, creep and relaxation.
Among the different time effects, only those acting during consolidation have been discussed here.
The available experimental data are able to clarify the following items:

1. Data from oedometer tests on specimens ranging from 76 to 508 mm in height performed by Mesri and Choi (1985) support the hypothesis that creep occurs only after the end of primary consolidation, i.e. after dissipation of excess pore pressure.
2. Field cases wherein settlements continue while the excess pore pressures remain constant (observed in highly structured clays) can be explained by nonlinear compressibility, rather than by creep phenomena.
3. Experimental data by the writers show that $K_o$ is practically constant during secondary compression. Consequently the views expressed by Kavazanjian and Mitchell (1984) (i.e. $K_o$ increases with time for low OCR clays and decreases for OC clays with $K_o$ greater than 1.0) either do not apply to all cohesive soils or are premature.

# 3. IN SITU TESTING

## 3.1. INTRODUCTION

The topics of this chapter cover only some selected aspects of the in situ tests and their interpretation.
Readers who are interested in going deeper into the most recent applications of in situ tests are invited to consult the following publications: Mitchell et al. (1978), Schmertmann (1978), Baguelin et al. (1978), Marchetti (1980), Baligh et al. (1978), Lacasse et al. (1978), Marchetti and Crapps (1981), Robertson and Campanella (1984), Mair and Wood (1985), and Meigh (1985).
This chapter will deal with the following specific problems:

### I. ASSESSMENT OF INITIAL STATE VARIABLES

#### Soil Profiling and Identification

The usefulness, the potential, as well as the limitations of different in situ techniques for continuous or quasi-continuous penetration of the soil will be discussed. These techniques make it possible to obtain detailed information on the soil profile, including the identification of different layers and the location of drainage boundaries. In this respect, the following in situ tests appear to be particularly useful:

1. Electrical Standard Friction Cone Test [CPT]; see De Ruiter (1971, 1981, 1982).
2. Piezocone (CPTU) [Janbu and Senneset (1974), Parez et al. (1976), Baligh et al. (1980), De Ruiter (1981, 1982)] and Pore Pressure Probe Test (PPP) [Torstensson (1975), Wissa et al. (1975)]. (The Piezocone features essentially a CPT tip which measures the cone resistance, $q_c$, the local lateral friction, $f_s$, and the pore pressure, $u_{max}$, generated during penetration. The Pore Pressure Probe measures $u_{max}$ only).
3. Marchetti Flat Dilatometer (DMT) [Marchetti (1980, 1982), Marchetti and Crapps (1981), Campanella et al. (1985)].

It should be recognized that there are other in situ techniques like the Electrical Resistivity Probe [Kutter et al. (1979), Arulmoli et al. (1981)], the Acoustic Cone [Villet et al. (1981), Tringale and Mitchell (1982)] and the Seismic Cone [Robertson and Campanella (1984)], all of which may have a potential for soil profiling and identification. However, their use in practice is at present limited due to a lack of adequate field experience. Thorough laboratory and in situ calibration of these devices is therefore recommended.

#### Measurement of Existing In Situ Horizontal Stress

It has long been recognized that the measurement or even an estimate of the initial horizontal in situ stress, $\sigma_{ho}$ or $\sigma'_{ho}$, is one of the most challenging tasks of ESE. Despite much effort during the last ten years, the problem still seems far from being satisfactorily solved. The main difficulties are due to the fact that none of the existing in situ devices can be inserted into the soil without disturbance, with the possible exception of the self-boring pressuremeter. Thus, the values of $\sigma_{ho}$ measured by the different instruments are compared to each other or to $\sigma_{ho}$ (or $\sigma'_{ho}$) from laboratory tests, without really knowing the correct result.

The following techniques will be examined in this section:

1. Self-Boring Pressuremeter Tests (SBP) [Baguelin and Jezequel (1973), Wroth and Hughes (1973)].
2. DMT [Marchetti (1980)].
3. Iowa Stepped Blade (ISB) [Handy et al. (1981, 1982)].
4. Spade-like Total Stress Cells (TSC) [Massarsch (1975), Massarsch et al. (1975), Ted and Charles (1981, 1983)].
5. Hydraulic Fracture Test (HFT) [Bjerrum and Anderson (1972), Marr (1974)].

Because insertion of most devices induces non-negligible strains, interpretation of the test results must incorporate explicitly or implicitly some empirical rules for the correction of the measured stress.

#### Assessment of the Stress History of the Soil Deposit

At present, there are no in situ techniques which allow the direct determination of the maximum past pressure.
Baligh et al. (1978) postulated that the simultaneous measurement of $q_c$ and $u_{max}$ during CPTU penetration may give an indication of the stress history of cohesive deposits. This possibility will be examined in the light of experience gained during the past five years with CPTU tests.

### II. DEFORMATION PARAMETERS

Given the limitations of the assessment of soil deformation parameters from laboratory tests, particularly when dealing with OC clays and cohesionless soils, their determination by in situ tests is considered to be of great practical interest.
The following in situ techniques, which are suitable for the determination of soil stiffness, will be briefly examined:

1. SBP [Wroth (1984), Mair and Wood (1985)], Ménard Pressuremeter (PMT) [Baguelin et al. (1978)], and the recently developed Push-In Pressuremeter, PIP [Henderson et al. (1979)].

2. CPT [Schmertmann (1972, 1978), Schmertmann et al. (1978), Senneset et al. (1972), Campanella and Robertson (1983), Robertson and Campanella (1984)].

3. DMT [Marchetti (1980), Boghrat (1982), Robertson and Campanella (1983)].

4. Plate Loading Tests (PLT), including Screw Plate Tests (SPL) [Janbu and Senneset (1973), Kay and Parry (1982), Marsland (1971)].

Mainly due to space limitations, the writers did not make an evaluation of the seismic methods, which through measurements of the propagation velocity of elastic waves (especially shear waves $v_s$) in supposedly horizontally layered deposits permit the shear modulus $G_{max}$ at small shear strain level ($\gamma \cong 10^{-5}$%) to be assessed [Woods (1978)].
Despite this omission the writers believe that methods such as the measurement of $v_s$ by the down-hole and especially cross-holes techniques [Miller et al. (1975), Bodare and Massarsch (1982), Nazarian and Stokoe (1984), Hoar and Stokoe (1984)] are of great relevance to the study of soil deformation moduli in situ, and they provide insight into the soil stress-strain behaviour at low strain levels.

## III. FLOW AND CONSOLIDATION CHARACTERISTICS

In view of the major role which in situ tests have played in the evaluation of flow and consolidation characteristics of natural soil deposits, the following testing techniques will be considered:

1. Borehole permeability tests [Hvorslev (1951)].

2. Pumping tests from wells with observation piezometers [Mansur and Kaufman (1962); Cassan (1980)].

3. Permeability tests run through piezometers [Hvorslev (1951), Gibson (1963), Wilkinson (1967, 1968)].

4. Self-Boring Permeameter Tests [Jézéquel and Mieussens (1975)].

5. Holding Tests using the SBP [Clarke et al. (1979)].

6. CPTU dissipation tests [Baligh and Levadoux (1980), Torstensson (1975)].

7. Evaluation of consolidation characteristics on the basis of pore pressure and strain measurements made under loaded areas [Bishop and Al-Dhahir (1970), Asaoka (1978), Orleach (1983), Jamiolkowski and Lancellotta (1984)].

The possibilities offered by these relatively well-known tests and their limitations will be examined with particular reference to the following problems:

1. Which type of flow conductivity (k) or consolidation (c ) characteristics may be obtained from each of these tests when dealing with an anisotropic and/or non-homogeneous soil deposit?

2. Since both k and c depend on the effective stress level, what is the significance of the parameters obtained from the above tests in relation to the existing effective in situ stress system and a given stress history?

### 3.2. ASSESSMENT OF THE INITIAL STATE VARIABLES (SOIL PROFILING AND IDENTIFICATION; INITIAL IN SITU STRESS, STRESS HISTORY)

### 3.2.1. Cone Penetration Test (Electrical Type Only)

In the late 1960's the CPT was greatly improved by the introduction of the electrical penetrometer [De Ruiter (1971)] which permits a continous measurement of both the cone resistance $q_c$ and local shaft friction $f_s$. Both parameters are sensed by two electrical strain gauge load cells, as shown in Fig.26. A detailed description of electrical penetrometers is found in De Ruiter (1981, 1982), Schaap and Zuidberg (1982) and Robertson and Campanella (1984). Its potential has been largely improved due to recent advances in data acquisition systems (see Chapter 4.) that allows the measured data to be obtained and stored in both graphical and digitized forms [De Ruiter (1982), Bruzzi and Cestari (1983), Robertson and Campanella (1984)]. The latter is extremely useful for most engineering applications of CPT and CPTU results.

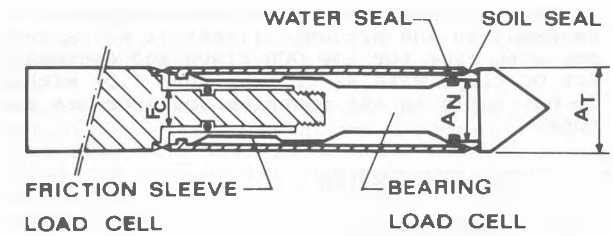



Fig.26: Typical Electrical Cone [Schaap and Zuidberg (1982)].

Because of improvements in equipment, testing procedures, and interpretation, the electrical CPT has become the major tool for offshore soil investigations [De Ruiter and Richards (1983)]. It presently offers an almost exclusive capability for obtaining reliable information on soil deposits in deep water.
The electrical cone has been tentatively standardized [ISSMFE (1977) and ASTM (1979)], and at present it has the following relevant features:
- cone with an apex angle 60° and base area of 10 cm$^2$;
- friction sleeve located immediately behind the cone having an area of 150 cm$^2$.

Through continuous monitoring of $q_c$ and $f_s$ and their ratio $FR = q_c/f_s$ (called the friction ratio), many attempts have been made to develop classification charts relating the soil type to the measured $q_c$ and FR [Begemann (1965), Schmertmann (1978)].
When using such charts and generally when interpreting CPT and CPTU results the following factors must be kept in mind:

1. It has been realized [De Ruiter (1981, 1982); Campanella and Robertson (1983)] that, when the electric cone is subjected to hydrostatic water pressure, an important shift of the reference electronic signal (zero readings) is observed for both the load cells incorporated in the tip (see Fig.26).
This phenomenon is due to the unequal areas of both the cone and the friction sleeve on which the water pressure is acting (Fig.27). Thus, both $q_c$ and $f_s$, as measured during the penetration process, do not represent the total resistance offered by the surrounding soil. They are

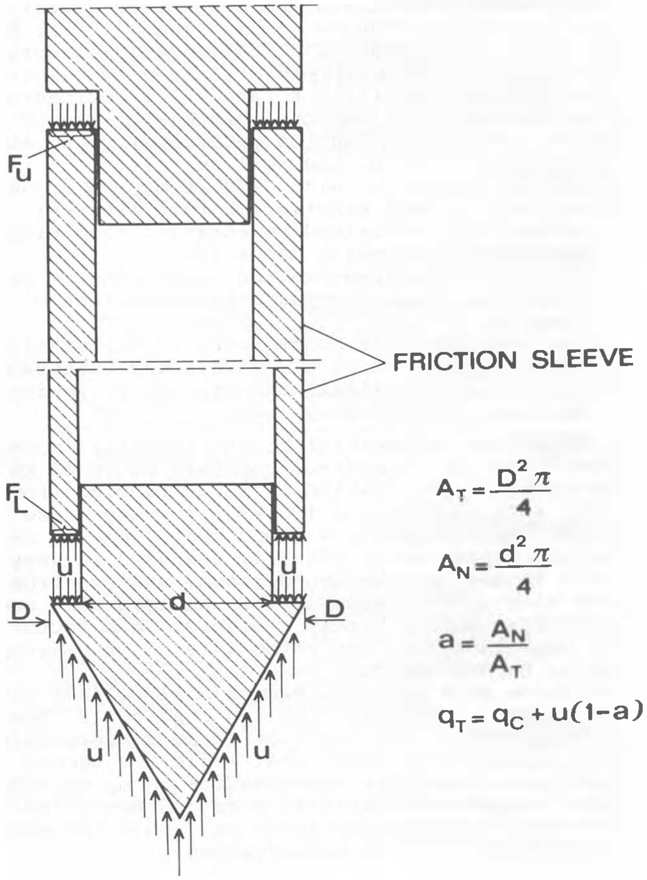



Fig.27: Unequal End Areas of the Electrical Friction Cone.

somewhat lower, depending on the specific construction of the cone.
In order to convert the measured values of $q_c$ and $f_s$ into the corrected ones to be expressed in terms of total stresses $q_t$, $f_t$, it is necessary to know the area ratios for the cone $a = A_N/A_T$ and for the sleeve $b = F_L/F_U$, respectively, as indicated in Fig.27.
These values should be obtained through careful laboratory calibration; see the example in Fig.28. The electrical cones presently in use have values of a between 0.75 and 0.85; however for some cones, these values may be as low as 0.5.
The correction of $q_c - f_s$ into $q_t - f_t$ values discussed herein requires the penetration pore pressure to be measured, and this is possible only when the CPTU is performed.
In this case:

$$q_t = q_c + k_c \; (1 - a) \; u_{max}$$

$$f_t = f_s + k_s \; (1 - b) \; u_{max}$$

where:
$k_c$, $k_s$ = correction factors depending on the offset between the point where $u_{max}$ is measured and the base of the cone and mid-height of the friction sleeve respectively [see Lunne et al. (1985)];
$u_{max}$ = measured penetration pore pressure.

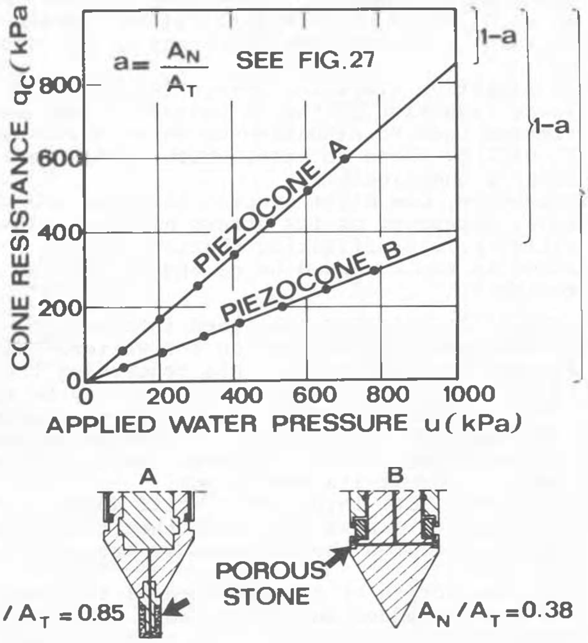



Fig.28: An Example of Determination of $A_N/A_T$ in the Pressure Chamber [Battaglio and Maniscalco (1983)].

Unfortunately, these corrections are not straightforward since the $u_{max}$ is not constant along the penetrometer shaft as shown in Fig.32. In fact it depends on the offset between the points in the ground at which the $q_c$ and $f_s$ and $u_{max}$ are measured. When $u_{max}$ is monitored only at one location, the correction of both $q_c$ and $f_s$ may be done by relating the measured pore pressure to the theoretical distribution along the penetrometer tip and shaft, as shown in Fig.32. Measurements of $u_{max}$ at the cone base allow a direct correction of $q_c$.
The unequal areas in the cone (Fig.27) have an important influence on the measured $q_c$ and $f_s$ values in cohesive soil deposits. This is especially true in soft clays, where high $u_{max}$ and low $q_c$ and $f_s$ lead in some cases to paradoxes like measured $u_{max} > q_c$ and negative values of $f_s$.
Once the importance of the above problem has been admitted, it must also be recognized that all existing correlations of soil type with $q_c$ or FR, have been developed for uncorrected $q_c$ and $f_s$ values. Therefore in the case of cohesive soil deposits, these correlations incorporate systematic errors, depending on a combination of specific features of the cone design and characteristics of the soils used for their formulation.
In view of the development of the CPTU which allows a correction of both the $q_c$ and the $f_s$ to be readily made, the preparation of improved soil identification charts based on corrected $q_t$ and $f_t$ values appears to be warrented.

2. When using the FR for identification purposes, it must be kept in mind that both $q_c$ and $f_s$ are influenced by the inital effective lateral stress $\sigma'_{ho}$ existing in the ground [Schmertmann (1972)], a parameter which reflects strongly the stress history of the penetrated deposit.

On the basis of the CPT performed in sands in calibration chambers [Schmertmann (1978), Baldi et al. (1983)], it appears that the effect of changes in $\sigma'_{ho}$ is more important on $f_s$ than on $q_c$.
In sensitive clays the severe remoulding and large reduction of the $\sigma'_h$ caused by cone penetration lead to situation in which the measured $f_s$ will be close to zero, which makes the FR largely questionable.
Therefore, one might suspect that for the same soil, depending on its stress history and sensitivity, classification charts like the one shown in Fig.29 based on $q_c$ and FR will be unreliable.

3. Based on existing published information [Schmertmann (1978)] and on the writers' experience, the following points concerning the use of CPT in stratified soil deposits may be made:
   . The thickness of thin stiff layers embedded in the soft soil mass should exceed approximately 70 cm in order for the cone tip to achieve full $q_c$ at its mid height.
   . For thin soft layers in a stiff deposit, the minimum thickness to assure that the correct $q_c$ is measured should probably exceed 20 to 30 cm.
   . The detection of the presence of thin soft layers embedded in stiffer soil deposits requires digital output of $q_c$ and $f_s$ values at least every 2 cm. Even in this case the detection of soft lenses and layers whose thickness is less than 20 cm requires much expertise and experience.

4. It is necessary to recognize that the successful use of CPT results for soil profiling and identification requires standardization of the cone design and of calibration [Schaap and Zuidberg (1982), Robertson and Campanella (1984)] and testing procedures [Schmertmann (1978), Robertson and Campanella (1984)]. It is especially important for CPT interpretation to maintain the standard penetration rate ≅ 2 cm/sec because the measured soil response may be influenced by it, depending on the permeability and strain rate sensitivity of the soil deposit.
   Based on experience with CPTU it may be argued that the standard penetration rate assures:
   . practically undrained penetration conditions in homogeneous cohesive deposits;
   . almost drained penetration conditions in relatively clean sands having a fines content of less than 10%.
   For this specific point refer to Roy et al. (1982), Bellotti et al. (1983) and Robertson and Campanella (1984) and see the following section.

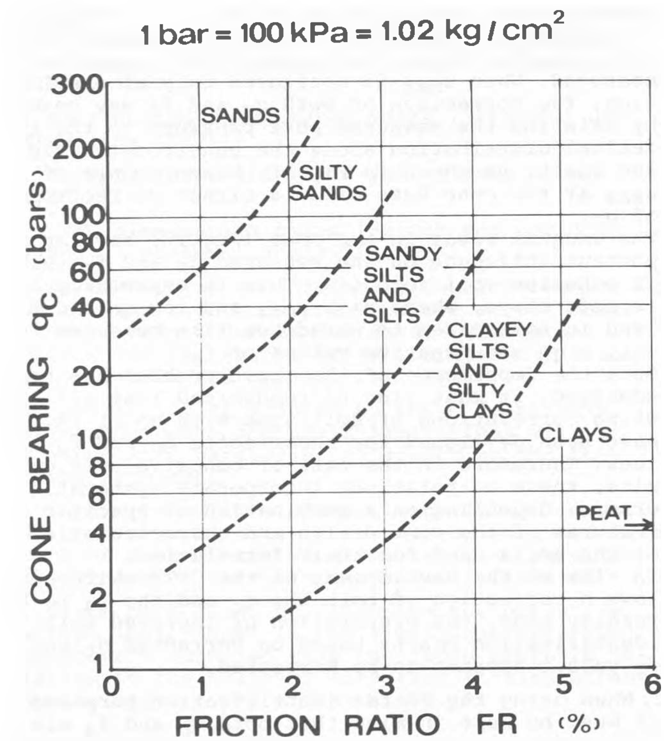



Fig.29: Simplified Classification Chart for Standard Electrical Friction Cone [Robertson and Campanella (1984)].

5. Almost all electrical cones presently in use have load cells (maximum capacity 50 to 80 kN) which allow penetration of soils ranging from very soft clays ($q_c \leq 100$ kPa) to very dense sands ($q_c$ frequently $\geq 30000$ kPa). Thus it is evident that the $q_c$ actually measured in very soft to medium cohesive deposits suffers from low electronic resolution of the load cell and therefore may be barely reliable. This factor is important when any ratio like FR or $u_{max}/q_c$ is to be evaluated.
   To solve this problem, Ridgen et al. (1982) have developed a cone tip with two load cells. One of them is capable of carrying loads corresponding to a maximum $q_c = 50000$ kPa, while the second much more sensitive one measures $q_c$ up to 5000 kPa. A specially designed overload mechanism protects the more sensitive cell once its maximum design load has been reached.

### 3.2.2. Soil Profiling and Identification from CPTU

The importance of pore pressure monitoring during the cone penetration was recognized in the mid seventies [Janbu and Senneset (1974), Schmertmann (1974)]. This was followed by the development of the Pore Pressure Probe (PPP) [Torstensson (1975), Wissa et al. (1975)] and the Piezocone [Janbu and Senneset (1974), Parez et al. (1976)]. Since then, a rapid development of the PPP and CPTU has occurred, which is evidence of the great potential that these devices have in soil exploration.
In the present paper attention is devoted only to the piezocone (Fig.30) because this device measures simultaneously $q_c$, $f_s$ and the penetration pore pressure $u_{max}$. From the interpretation point of view, this fact presents a distinct advantage over the PPP which allows the measurement of $u_{max}$ only.
The most interesting applications of the CPTU in geotechnical engineering are:

1. Soil profiling and identification; this topic will be briefly discussed in the present section.
2. Tentative assessment of the stress history of cohesive deposits; see section 3.2.9.
3. Evaluation of the flow and consolidation properties in cohesive deposits; see section 3.4.5.
4. Assessment of ground water conditions [Battaglio et al. (1981), Robertson and Campanella (1984)].
5. Indication concerning the liquefaction susceptibility of sand deposits [Schmertmann (1978a), Norton (1983), Kok (1983), Robertson and Campanella (1984a)].

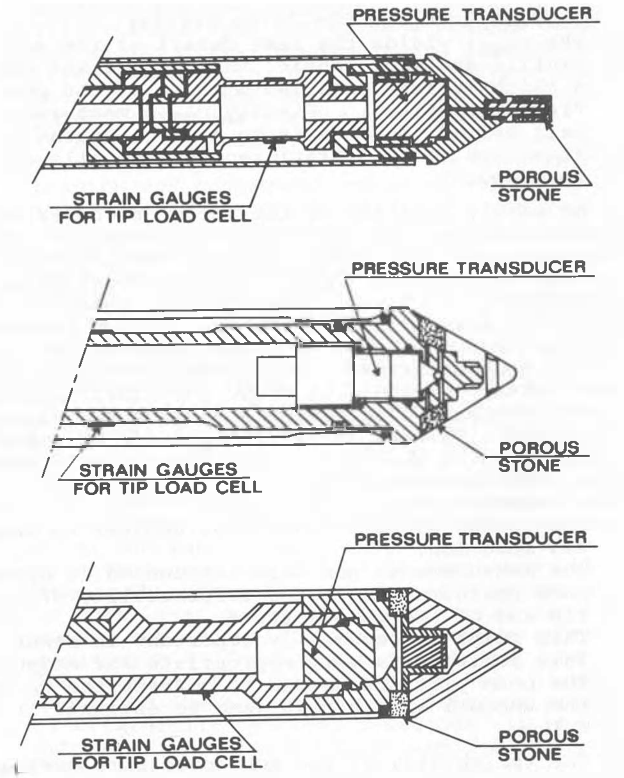



Fig.30: Example of Existing Piezocones.

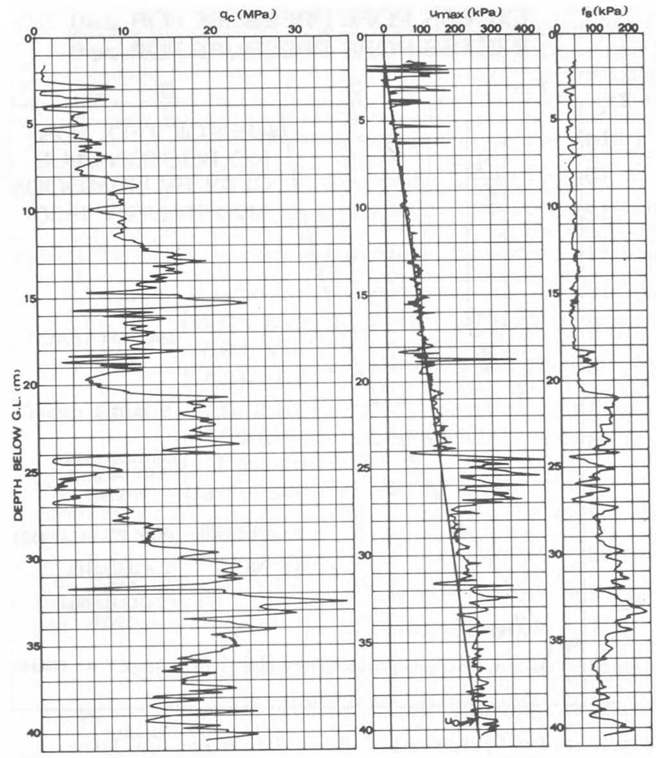



Fig.31: Results of CPTU Performed in Recent Alluvial Deposits of the Po River Valley [Battaglio and Bruzzi (1984)].

In addition to the above mentioned applications, a knowledge of $u_{max}$ rationalizes and makes less empirical the interpretation of $q_c$ in both cohesive and cohesionless deposits. This makes possible:

1. An assessment of the drainage conditions under which penetration is performed. The experience gained in the last five years has showed that in clays and uniform silts, cone penetration at the standard rate of 2 cm/sec occurs virtually in undrained conditions [Roy et al. (1982, 1982a), Campanella et al. (1981), Battaglio and Bruzzi (1984)]. In clean sands (< 10% passing ASTM sieve No.200), the penetration of CPT at the standard rate is virtually a drained process [Smits (1982), Bellotti et al. (1983)]. This is well supported by the results of CPTU performed in the recent alluvial deposits of the Po River Valley shown in Fig.31. At a depth between 4 and 24 meters below G.L. the soil consists of medium to coarse dense sand with from 2 to 10% fines. In this layer the cone penetration occurs with a negligible variation of the pore pressure with respect to the equilibrium $u_o$ line.
2. The interpretation of $q_c$ in both cohesive and cohesionless deposits by adopting a more fundamental effective stress approach [Janbu and Senneset (1974), Senneset et al. (1982), Senneset and Janbu (1984)].
3. The correction of $q_c$ and $f_s$ for the unequal areas effect.

Despite the great potential of CPTU, experience accumulated in the last decade with CPTU and PPP strongly suggests that reliable and accurate measurements of $u_{max}$ during cone penetration are still difficult and controlled by a number of factors which have scarcely been investigated and thus are not completely understood. The factors are (1) a comprehensive and generally valid theory for the prediction of the stress and strain fields, hence of the $u_{max}$ around the penetrating cone, (2) equipment design criteria, and (3) test procedures [see Levadoux and Baligh (1980), Rad (1983), May (1983), Robertson and Campanella (1984), Baligh (1984), Narayanan et al. (1984)]. These factors deserve some discussion, since they are inherently related to the use of piezocones in soil profiling and identification.

1. Baligh and co-workers [Baligh et al. (1978, 1980), Baligh and Vivatrat (1978), Levadoux and Baligh (1980)] demonstrated both theoretically and experimentally, that $u_{max}$ depends on the position of the porous stone on the penetrometer tip. The theoretical distribution of the penetration excess pore pressure predicted by Levadoux and Baligh (1980) for the OCR=1 Boston Blue clay is compared to experimental data obtained by different researchers in Fig.32. The theoretical predictions agree quite well with the measurements.
2. Fig.33 reports some of the values measured at the Pontida site in Italy which is a deposit of medium stiff silty-clay with a highly developed macro-fabric. The measurements have been performed with a PPP having three porous sensors located as shown in Fig.33.

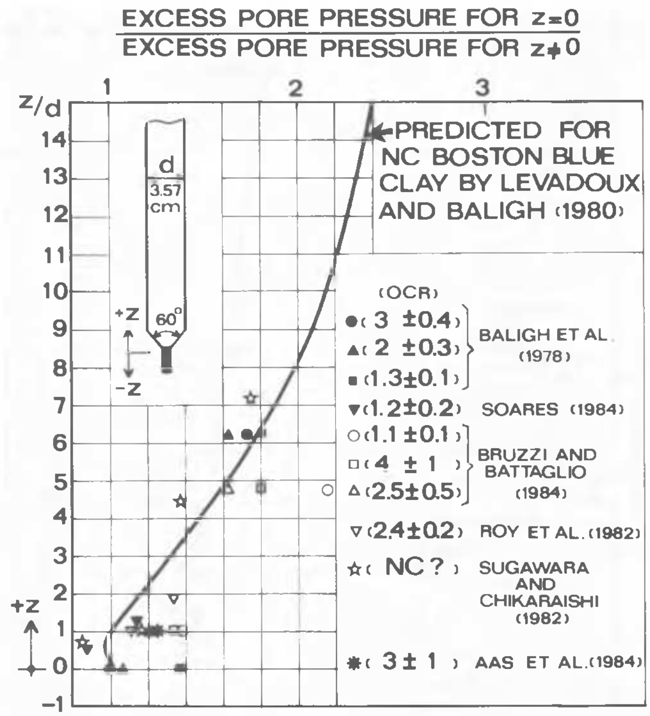



Fig.32: Influence of Filter Location on Pore Pressure Measured During Penetration for 60° Standard Cone.

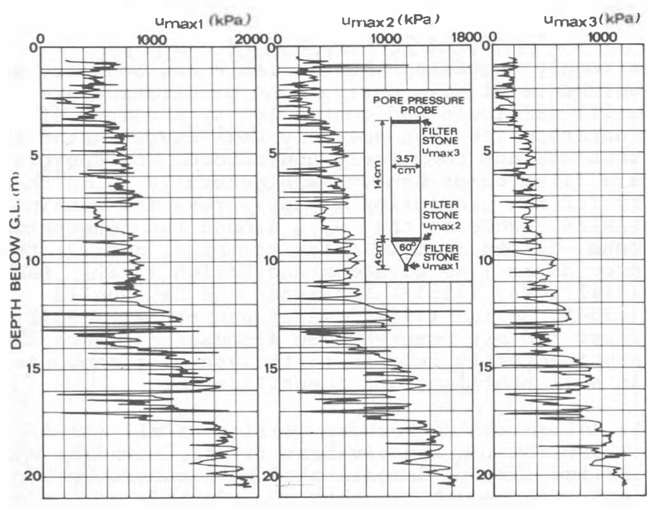



Fig.33: Penetration Pore Pressure in a Medium to Stiff Clay at the Pontida Site. [Battaglio and Bruzzi (1984)].

An analysis of this test and similar data enables the following practical remarks to be made:

- The $u_{max1}$ measured at the cone apex is 17 to 26% higher than the $u_{max2}$ measured immediately behind the cone, and it is 50 to 60% higher than the $u_{max3}$ measured 14 cm above the cone.
- The ratios of $u_{max1}/u_{max2}$ and $u_{max1}/u_{max3}$ seem to be slightly influenced by the value of OCR in the sense that they increase as the OCR decreases (compare Fig.33 to Fig.59).
  The $u_{max1}$ yields the best detail of the soil profile and macrofabric. The $u_{max2}$ gives only a slightly less detailed picture of the penetrated deposit. Finally $u_{max3}$ measured well behind the cone is to a large extent less sensitive to the stratigraphical details of the deposit.

3. No single location of the filter stone exists which provides the "best" pore pressure for all the possible applications of the CPTU. However, as far as soil profiling and identification are concerned, it appears that the best position for the filter stone is the one located somewhere on the cone. On the other hand, considering the still good stratigraphical details obtained with the filter located just above the cone, there are some reasonable arguments [Robertson and Campanella (1984), Tavenas et al. (1982)] for this position also, if the main aim of CPTU is soil profiling and identification.
   The arguments are:
   - The porous stone is much less subject to damage and abrasion.
   - The measurements are less influenced by dynamic pore pressure due to the deformability of the tip and of the porous stone.
     This point is especially important in sands.
   - This position is very appropriate for measuring the pore pressure needed to correct the $q_c$ for the unequal area effect (and to estimate it for $f_s$).
4. The reliability of the monitored pore pressure depends largely on the rigidity of the measuring system.
   For properly designed piezocones this is mainly controlled by the quality of de-airing. Campanella et al. (1981) and Battaglio et al. (1981) have presented experimental data indicating the importance of appropriate de-airing if reliable information both during the penetration and dissipation stages of CPTU is to be obtained. Experience [Baligh et al. (1981), Acar (1981), Franklin et al. (1981), Lacasse and Lunne (1982a), Rad (1983), Battaglio and Maniscalco (1983)] indicates that the de-airing and assembly of piezocones should preferably be performed in the laboratory under high vacuum in specially designed chambers with thoroughly de-aired saturation fluids. Water, glycerin, and silicone oil can be used. After the completion of the de-airing operation, the tip is enclosed in a special plastic container filled with the saturation fluid. It can then be transferred to the testing site without losing saturation.
   Many research organizations check dynamic time response of CPTU tips after their saturation and after the test, see Fig.87.
5. Very little is known about the criteria concerning the selection of the filter element to be used in CPTU. The effects of the filter type, dimension, permeability, and compressibility are scarcely understood. Based on the writers' experience and that of Smits (1982), Rad (1983), Battaglio and Bruzzi (1984), the following points may be made about filters:
   - In cohesive deposits a high entry value fine graded filter should be used.
     According to Smits (1982), a 2 μ pore diameter porous stone, saturated with silicone oil, leads to a correct pore pressure response under a cyclic loading of 100 Hz. This kind of porous sensor can possibly penetrate unsaturated soils without losing its saturation.

- Abrasion and the consequent clogging of the filter stone seem not to be a problem in soft and medium clays. However these problems can arise when testing OC clays, and therefore in such deposits the use of the filters made of porous stainless steel may be advisable.
- In sands, especially in those with high $q_c$ and predominantly hard minerals, the problem of abrasion and consequent smear and clogging controls the selection of the filter materials [Smits (1982)]. The experience gained in testing these kinds of materials has shown that, depending on the density and mineralogical composition of the soil and on the mechanical characteristics of the filter element, abrasion from penetration can cause complete or partial clogging of the porous sensor, rendering the measured pore pressure almost completely unreliable. Fig.31 shows the already mentioned CPTU run in the alluvial deposits of the Po River Valley. It was observed that during penetration of the sand layer above a depth of 24 m, the measured $u_{max}$ matches closely the $u_o$. But this is not true in the sand below 27 m. Analyses of soil samples showed that this sand contains generally less than 10% fines. Therefore the trend $u_{max} > u_o$ observed in this layer makes the experimental data suspect, and indicates possible porous stone clogging or self-deformability of the cone tip. The example of this is shown in Fig. 34 which shows the instantaneous pore pressure response of a carefully de-aired piezocone tip when subjected to external loads applied respectively to the cone face ($P_2$) or to the porous stone ($P_1$). In the presence of cyclic loading during unload, a negative $\Delta u$ is also generated inside the cone. This example emphasizes the need of a very rigid piezocone tip design when testing soils with high $q_c$.

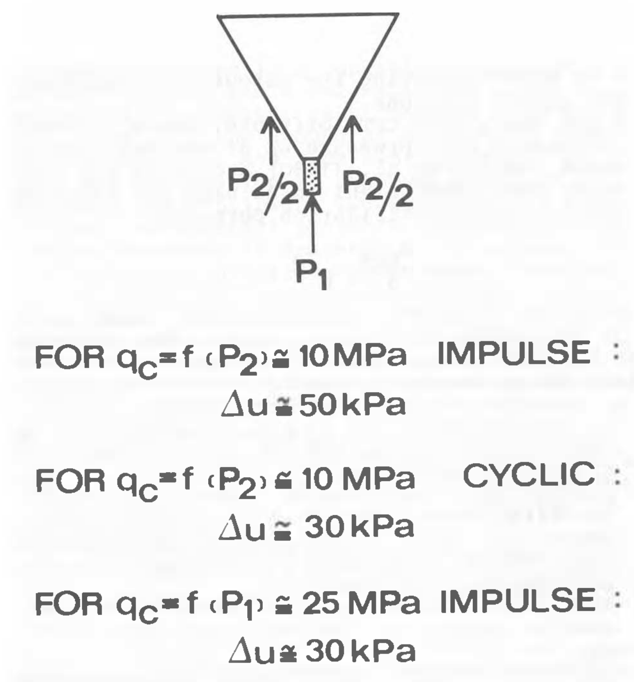



Fig.34: Excess Pore Pressure Generated by the Deformability of the Cone [Battaglio and Bruzzi (1984)].

In this respect Fig.35 presents the results of two CPTU performed in a medium to coarse sand with the silt and clay fraction on average between 2 and 6% but never exceeding 14%. The test run with the Delft piezocone designed by Smits (1982) having the porous stone made of specially hardened sinterized steel yielded $u_{max}$ values which close to or only slightly below the equilibrium $u_o$ line.
On the contrary the tip with the porous stone at the apex led to $u_{max}$ values which were generally well above the hydrostatic line yielding positive excess pore pressure varying between 50 and 100 kPa.
The difference in the measured pore pressures may be due to a combination of factors such as:
. Differences in the total stress fields generated at the filter locations.
. Abrasion and smear of the porous stone located at the apex of the second piezocone.
. Axial deformability of the second cone tip.

The above remarks cover briefly some of the complex problems involved in the appropriate design and calibration of piezocone tips. More details concerning these problems may be found in the work by Zuidberg et al. (1982), Schaap and Zuidberg (1982), De Ruiter (1981, 1982) and Robertson and Campanella (1984).

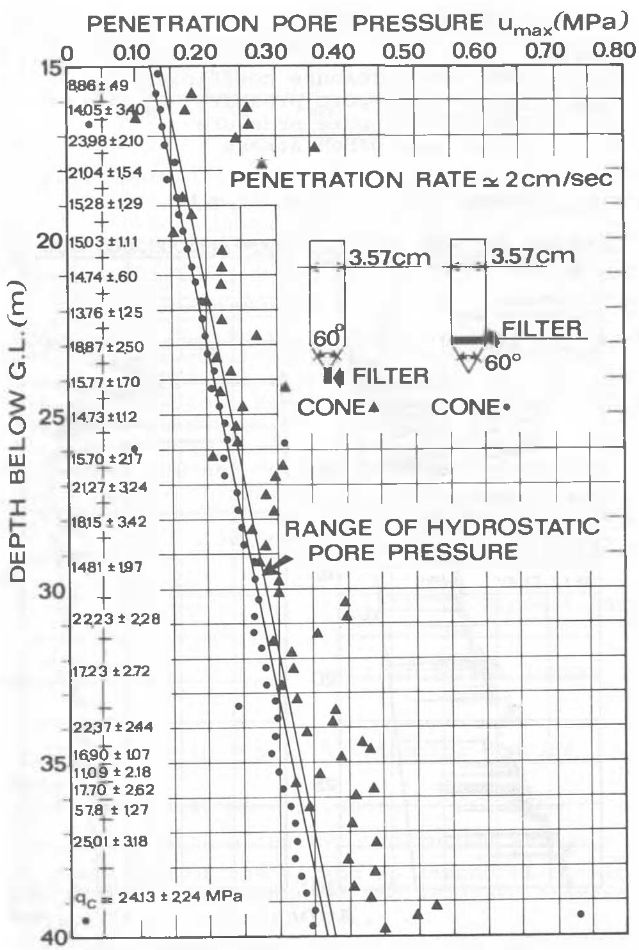



Fig.35: Comparison Between Penetration Pore Pressures Obtained with Two Different Piezocone Tips in NC Dense Sand.

Despite the above mentioned difficulties it is certain that a properly designed, calibrated, and thoroughly de-aired piezocone represents the most sensitive tool presently available for soil profiling, detection of macro-fabric and drainage boundaries; see example shown in Fig.36 and 33. The piezocone enables thin lenses and layers of different soils to be detected with a much higher degree of accuracy than with the CPT. The experience gained at different research organizations indicates that with a properly de-aired tip combined with an adequate data acquisition system, the detection of soil layers only few centimeters thick becomes possible. CPTU appears also to be an excellent tool for soil identification since the ratios $u_{max} - u_o/q_t - \sigma_{vo}$, $\Delta u/\sigma'_{vo}$, etc. are sensitive to the nature of the penetrated soil [Baligh et al. (1978, 1980, 1981), Jones and Rust (1982), Campanella et al. (1982, 1982a), Tumay et al. (1981, 1982), Robertson and Campanella (1983, 1983a, 1984), Senneset et al. (1982), Senneset and Janbu (1984)]. Among the large number of the CPTU pore pressure ratios proposed by different authors as a soil properties index [see the review by Robertson and Campanella (1984)], the one used by Senneset et al. (1982) seems to have the best theoretical basis [Wroth (1984)]. It is defined as:

$$B_q = \frac{u_{max} - u_o}{q_t - \sigma_{vo}}$$

where:
$B_q$ = CPTU pore pressure coefficient
$u_{max}$ = penetration pore pressure
$u_o$ = hydrostatic pore pressure
$\sigma_{vo}$ = total overburden stress

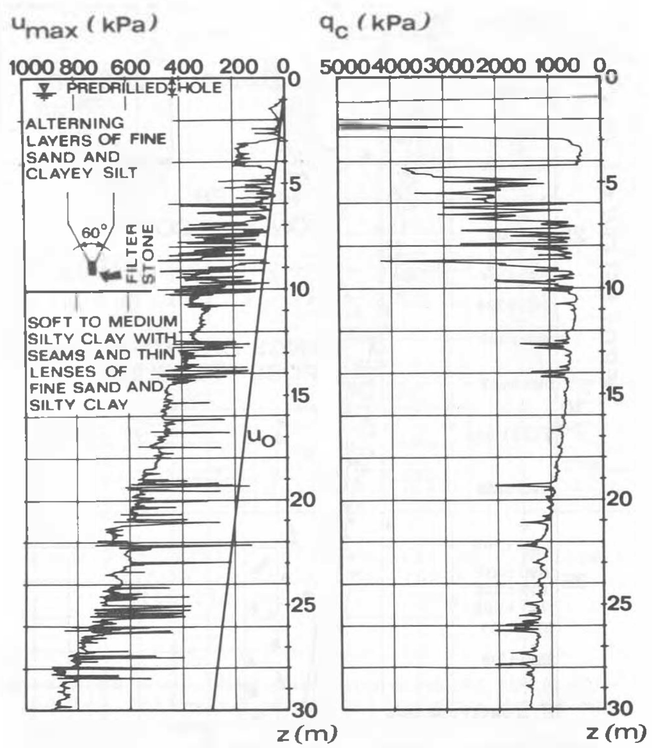



Fig.36: Example of CPTU Performed in a Cohesive Formation with Highly Developed Macrofabric, Porto Tolle (Italy)
[Battaglio and Maniscalco (1983)].

$q_t$ = total cone resistance corrected for the unequal end area effect

Using this parameter, Senneset et al. (1982) and Senneset and Janbu (1984) presented the tentative soil classification chart shown in Fig. 37.

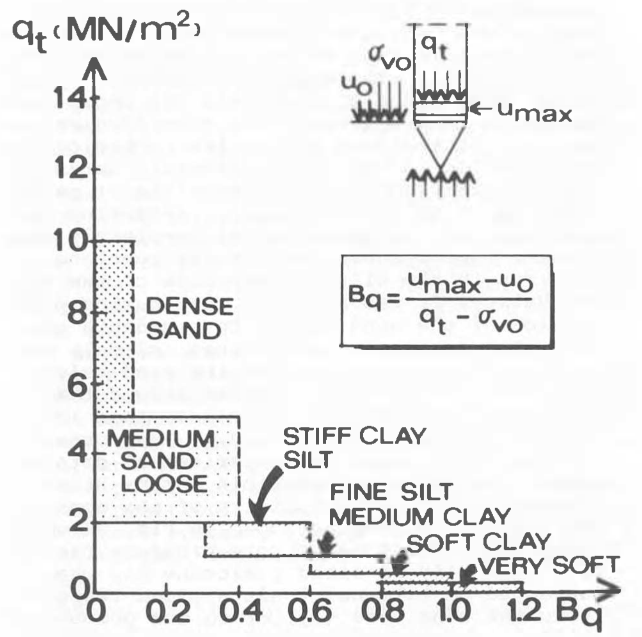



Fig.37: Tentative Classification Chart Based on $q_c$ and $B_q$ for Standard Electrical Friction Cone
[Adapted from Senneset and Janbu (1984)]

It is based on having the porous stone located just behind the cone.
In the use of the CPTU offshore, where a large hydrostatic pore pressure $u_o$ at the mud line exists, Baligh et al. (1980) suggested the following CPTU coefficient to be used for soil correlation and identification purposes:

$$R = \frac{u_{max} - u_o}{q_t - u_o}$$

Azzouz et al. (1983) have also used $u_{max} - u_o/\sigma'_{vo}$ for soil identification purposes. When using any of the soil classification systems based on CPTU data in engineering practice, it is necessary to pay attention to the following points:

- All of the CPTU pore pressure coefficients are more sensitive than the CPT friction ratio to changes in soil type.
- Each of the classification charts available in the literature is applicable only with reference to a specific piezocone geometry and only with respect to a specific position where the pore pressure is measured.
- Soil classification systems based on the CPTU results should use the total cone resistance $q_t$.
- All pore pressure coefficients formulated for CPTU ($B_q$, $\Delta u/\sigma'_{vo}$, $\Delta u/q_c$, $\Delta u_{max}/q_c$) reflect not only the type of the soil but also its stress history and probably also its stiffness to strength ratio.

### 3.2.3. Soil Profiling and Identification from Marchetti Flat Dilatometer (DMT)

The flat dilatometer was developed in Italy in the late 1970's by Marchetti (1975).
The dilatometer equipment is very simple. It consists of a thin blade (Fig.38) with a thin expandable steel membrane 6 cm in diameter on one face.

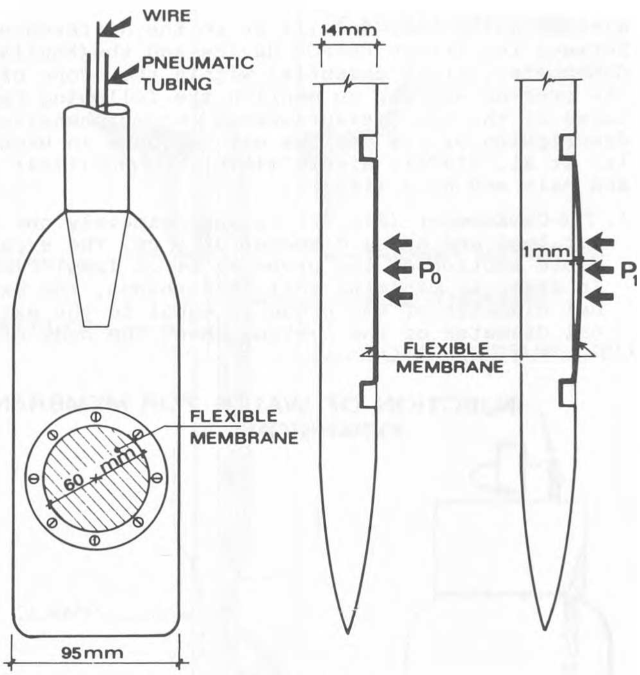



Fig.38: Marchetti Dilatometer.

In addition, there is a connecting cable, an air pressure source, and a read-out unit.
During the dilatometer test, the blade is pushed vertically into the ground at a constant rate, generally between 1 and 2 cm/sec, and every 10 to 20 cm the penetration is stopped and a test is performed. The following two readings are taken:

- First, the internal pressure causing lift-off of the membrane is determined. This value, after a number of appropriate corrections [Marchetti (1980), Marchetti and Crapps (1981)], yields the "first corrected reading" $p_o$.
- Second, the internal pressure necessary to expand the center of the membrane 1 mm is determined. This value after correction yields the "second corrected reading" $p_1$.

The two objectives of Marchetti's device were:

- to have a simple, highly repeatable and economic in situ device able to yield reliable information concerning soil stiffness, strength, and stress history; and
- to reduce as much as possible the shear and volumetric strains and nonuniformities caused in the surrounding soil by the insertion of the device.

The verification of the latter conditions has been attempted by Boghrat (1982) and Davidson and Boghrat (1983). These authors performed a series of laboratory tests in sand from which resulted that both volumetric and shear strains observed around DMT are appreciably lower and more uniform if compared to those occurring around the penetrated cone tip. On the basis of these experiments the authors concluded that "since the measured strains contours represent a measure of soil disturbance, the blade-shaped instrument, with measuring devices on the flat surface, appears to test a much less disturbed soil than do cone-shaped instruments".
Experience at MIT has been somewhat different. In situ tests were performed in clay with penetrometers and blades having the same volume.
In fact, data in Fig.39 [Baligh (1984a)] show the normalized values of the total radial stress $(\sigma_r - u_o) / \sigma'_{vo}$ as measured with three different blade types, compared with those (shown by the band) measured with the piezocone.

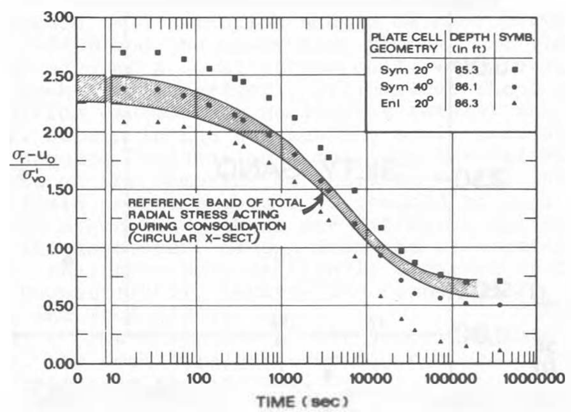



Fig.39: Normalized Values of the Total Radial Stress as Measured with Three Different Blades Compared with Similar Measurements with the Penetrometer [Baligh (1984-a, Personal Communication].

In spite of the very different aspect ratio (i.e. width versus thickness, equal to 12.5 for the blade and to 1 for the piezocone), the results are very similar; hence the same level of disturbance may be expected.
Marchetti (1980, 1982), Schmertmann (1983, 1984) and Robertson and Campanella (1984) have developed a number of useful empirical correlations allowing a rather comprehensive characterization of the penetrated deposit. These correlations refer to the following three index parameters:

1. $I_d = \dfrac{p_1 - p_o}{p_o - u_o}$ Material or Deposit Index

2. $K_D = \dfrac{p_o - u_o}{\sigma'_{vo}}$ Lateral Stress Index

3. $E_d = 38.2\ (p_1 - p_o)$ Dilatometer Modulus

where:

$u_o$ = in situ pore pressure

$\sigma'_{vo}$ = in situ effective overburden stress

On the basis of the $I_d$ and $K_D$ Marchetti developed a quite comprehensive and accurate identification chart for soil type.
The data reported by Boghrat (1982), Lacasse and Lunne (1982b), Schmertmann (1983, 1984), Robertson and Campanella (1983a, 1984) and Aas et al. (1984) showed that DMT provides nearly continuous data so that the type of the penetrated

soil can be identified with remarkable repeatability.
Boghrat (1981) developed a piezoblade. This device which has the geometry of the DM blade measures the pore pressure during penetration.
Based on the results of both DMT and piezoblade tests performed in Florida soils, Davidson and Boghrat (1982) proposed a classification chart (Fig.40) which enables the soil type to be determined on the basis of $I_d$ and of the percent of the $u_{max}$ dissipation occurring during a period of 1 min after the piezoblade penetration has been stopped.

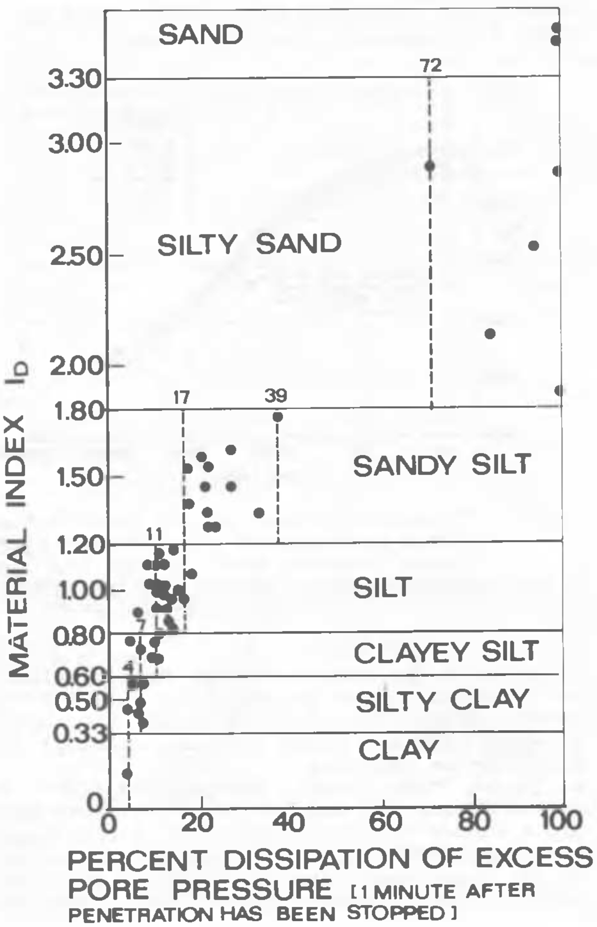



Fig.40: Tentative Classification Chart for Piezoblade [Boghart (1982)].

This type of correlation, which requires further validation, may improve the DMT potential for soil profiling, especially with the device developed recently by Campanella et al. (1985) which is able to measure simultaneously $p_o$, $p_1$ and $u_{max}$.

### 3.2.4. $\sigma_{ho}$ from Self-Boring Pressuremeter (SBP)

The self-boring pressuremeter (SBP) was developed simultaneously but independently in France [Baguelin et al. (1972, 1973, 1974)] and in the UK [Wroth and Hughes (1973, 1974), Hughes (1973)]. The attempt was to improve Ménard's pressuremeter test by eliminating or reducing the soil disturbance of a predrilled hole. It was recognized that the pressuremeter test, which simulates the expansion of a cylindrical cavity, is the only in situ device which works with well-defined boundary conditions and therefore permits a more rigorous theoretical analysis than for other in situ tests.
In both the French and English version SBP (Figs. 41 and 42), the same principle of self-boring was adopted.
Without going into details as to the differences between the French PAFSOR device and the English Camkometer, it is essential within the scope of the present section to mention the following features of the two pressuremeters [a comprehensive description of the devices may be found in Baguelin et al. (1978), Windle (1976), Clarke (1984) and Mair and Wood (1985)]:

1. The Camkometer (Fig.42) is approximately one meter long and has a diameter of 8 cm. The expandable section of the probe is 64 cm long.
   In order to minimize soil disturbance, the external diameter of the probe is equal to the external diameter of the cutting shoe. The body of

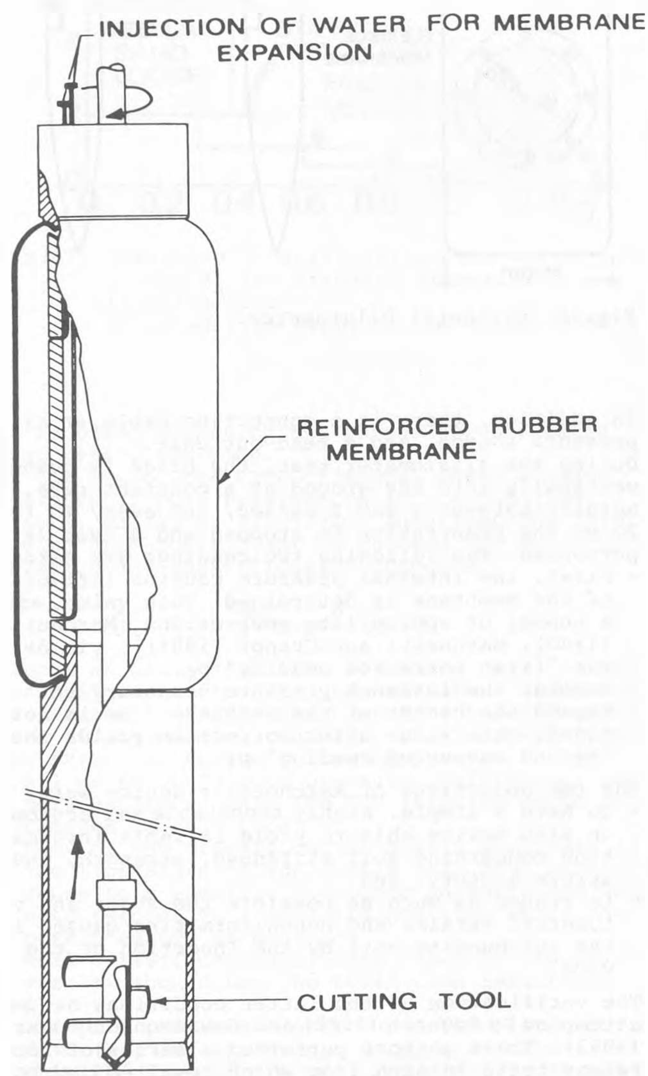



Fig.41: SBP Type PAFSOR.

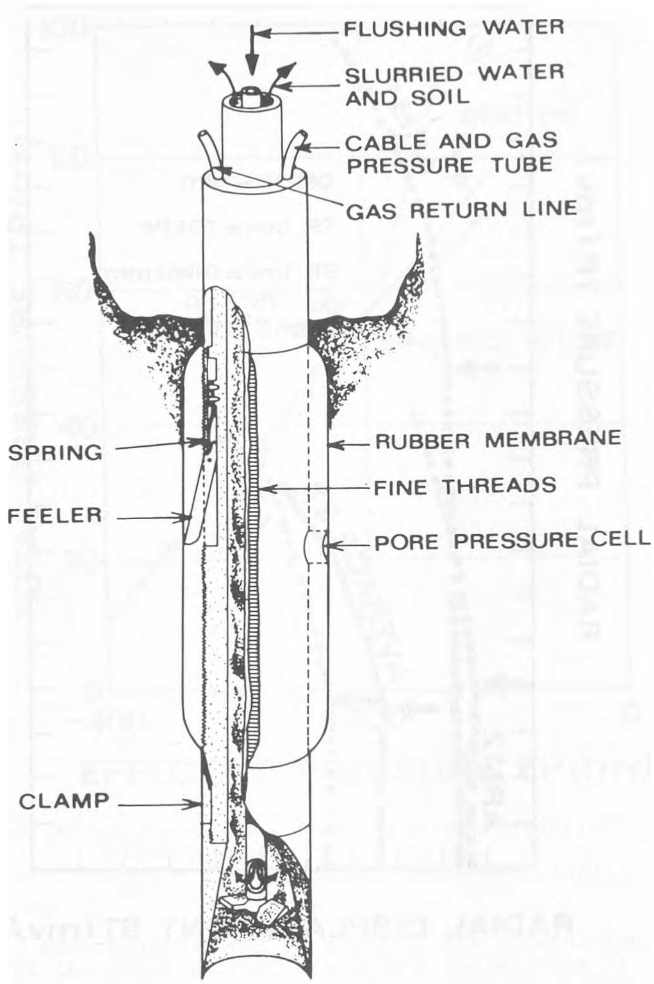



Fig.42: SBP Type Camkometer.

the probe is a rigid hollow cylinder so that the noninflated membrane, during its insertion in the ground, is forced to maintain a cylindrical shape.
After installation, the membrane, is inflated by gas pressure and its expansion is measured by three separate sensors (strain arms or feelers) placed at 120° around the midplane of the probe.
During the test, the pore pressure at the face of the membrane is measured, which means that the test yields the expansion p vs $\varepsilon_\theta$ or p vs $\Delta V/V_o$ curve in terms of both effective and total stresses,
where:

p = internal cavity pressure
$\varepsilon_\theta$ = the circumferential cavity strain, equal to the increase of the radius, $\Delta r$, divided by the initial radius, $r_o$
$\Delta V/V$ = volumetric cavity strain, with
V = initial ($V_o$) or current ($V_c$) volume of the expanded cavity, and
$\Delta V = V - V_o$

2. The PAFSOR probe (Fig.41) is approximately 215 cm long and has a diameter of 13.2 cm; the expandable section is 52.8 cm long.
Its internal rigid body has a smaller diameter than the cutting shoe so that the instrument is inserted with the membrane slightly expanded to an average diameter equal to the cutting shoe diameter.
Once the probe is inserted, the membrane is inflated by fluid injection and both the internal pressure and the inflated volume are monitored. The expansion curve is obtained either in terms of p vs $\Delta V/V$ or p vs $\varepsilon_\theta$. The strain is calculated assuming that the probe maintains a constant diameter which requires a uniform soil pressure.

For both devices it is essential to perform a series of calibrations in order to obtain a corrected expansion curve [see Baguelin et al. (1978), Windle (1976) and Mair and Wood (1985)].
As far as the assessment of $\sigma_{ho}$ from the SBP is concerned, a distinction should be made between the PAFSOR and the Camkometer probes. The PAFSOR probe acts as a total stress cell. Therefore, if the probe has a perfectly cylindrical shape and insertion causes only negligible lateral and vertical strain in the surrounding soil, then after an adequate "relaxation time", the lateral soil stress on the membrane as measured by the internal fluid pressure should correspond to $\sigma_{ho}$.
On the other hand, there are different options for the assessment of $\sigma_{ho}$ from the Camkometer test; they have been critically reviewed by Denby and Hughes (1982), Lacasse and Lunne (1982), and Mair and Wood (1985).
After a "low-disturbance" SBP insertion into the soil, the most straightforward procedure consists of monitoring the internal cavity pressure while increasing it incrementally in small steps from zero until the membrane just starts to expand.
The pressure at which this occurs is recorded separately by the three strain arm feelers which "lift off"; at that moment the "lift-off pressure" $p_o \cong \sigma_{ho}$, the external soil stress. This procedure is applicable to both clays and sands if there is negligible disturbance. In order to deduce with precision the lift-off pressure, it is necessary to examine the very initial part of the expansion curve on an enlarged scale (see Fig.43).
The lift-off pressure may be assessed from the average expansion curve of the three strain feeler readings or by examining the three strain arms separately which often yield three different values of $p_o$ [Dalton and Hawkins (1982), Benoit (1983), and Ghionna et al. (1983)].
The Fig.43(a) shows an example of the lift-off pressure as measured in soft clay at the Panigaglia site in Italy. In this case the difference in $p_o$ from the feeler arms is small; this is a typical situation encountered in soft cohesive deposits [see Ghionna et al. (1981, 1983) and Benoit (1983)]. On the contrary, in hard cemented clay at the Taranto site in Italy [Fig.43(b)], the difference in $p_o$ from feeler arms is very large. Similar experiences have been reported by Dalton and Hawkins (1982).
From experience gained at TUT in general the differences in $p_o$ from the feeler arms increase with increasing clay stiffness, and this phenomenon may be attributed to one or more of the following factors [Dalton and Hawkins (1982), Benoit (1983), Mair and Wood (1985), Ghionna et al. (1983)]:

- Noncircular shape of the pressuremeter hole and consequent nonsymmetrical disturbance of the soil.
- Mechanical compliance of the instrument itself.
- Deviation of the probe from the vertical.
- Nonuniform shear stress at the probe-soil interface.

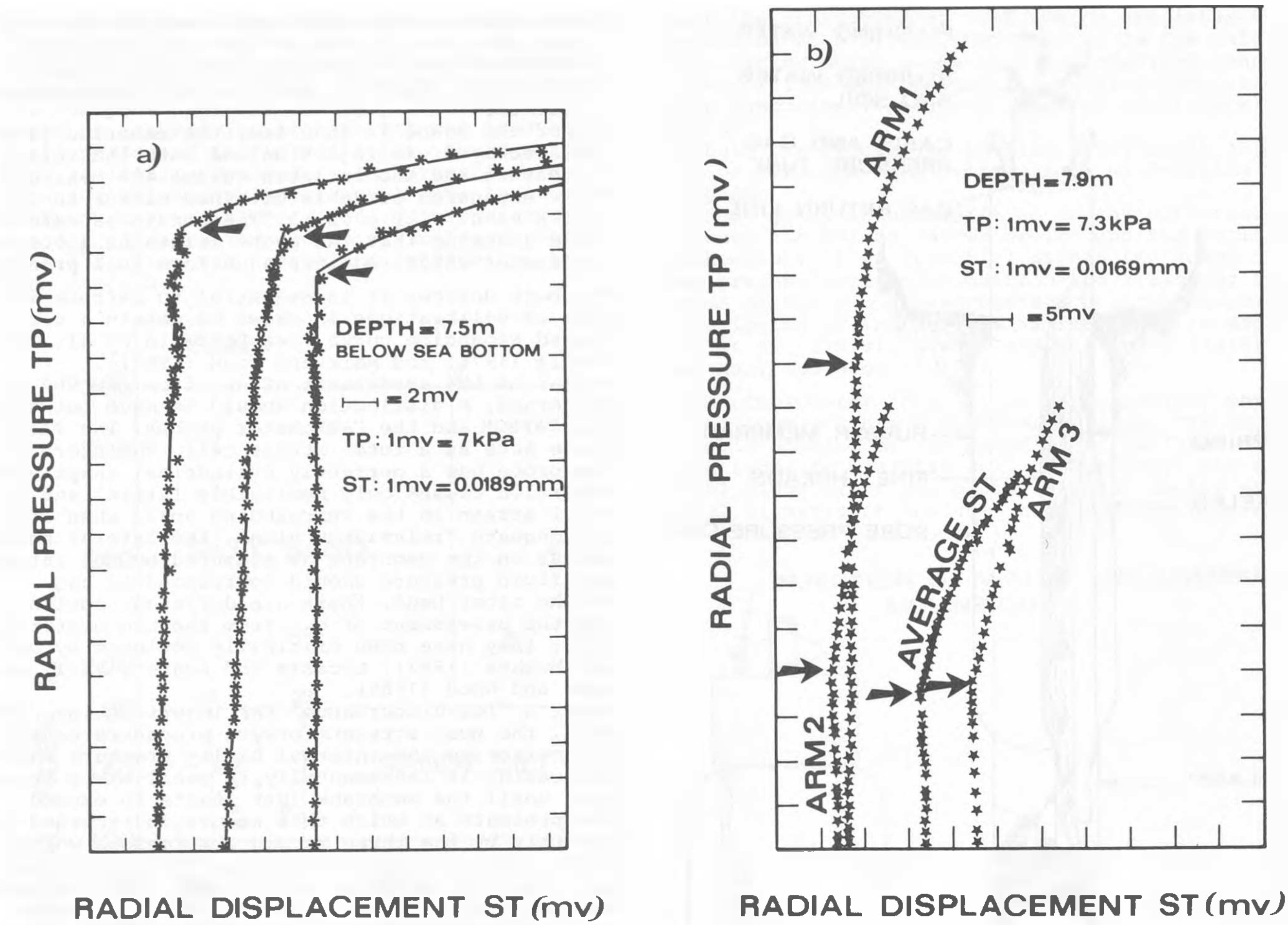



Fig.43: Example of Lift-Off Stress (a) In Soft Clay at the Panigaglia Site
(b) In Hard Clay at the Taranto Site.

- Anisotropy of the in situ horizontal stress. Certaintly this factor may occur in very stiff and hard clay deposits which have been subjected to significant tectonic movements and/or ice movements. But this is probably not the case for the Taranto clay [Ghionna et al. (1983)], and even less probable for the soft, recent cohesive deposits of Panigaglia and Porto Tolle [see Ghionna et al. (1983)].

Among the other available procedures for the evaluation of $\sigma_{ho}$ from the Camkometer which merit attention is the one proposed by Wroth and Hughes (1974) and Wroth (1982). The procedure assumes that $\sigma_{ho}$ is equal to the cavity pressure at which excess pore pressure at the cavity wall first develops (Fig.44). Obviously, this procedure is applicable to clays only, and particularly to clays of soft to medium consistency where a prompt and nonnegligible positive pore pressure response is usually observed immediately after p exceeds the total in situ stress.

Marsland and Randolph (1977) developed an iterative procedure based on the theory of the undrained expansion of a cylindrical cavity in an elastic, perfectly plastic soil. However in the writers' opinion, this procedure is applicable only to those OC lean clays which, at small strains, show a behaviour which is at least approximately similar to the one postulated.

Based on almost ten years experience with the SBP [Ghionna et al. (1982, 1983)], the writers conclude that rather than develop new interpretation procedures or improve the existing ones (which might lead to some kind of "manipulated" correction of the experimental data), the only reliable approach to the evaluation of $\sigma_{ho}$ from the SBP lies in using the "lift-off" approach. But to accomplish this, work must continue on improving the mechanics and electronics of the strain arms and on the insertion procedures. A reliable determination of $\sigma_{ho}$ is essential, not only for its own sake, but also in order to obtain reliable soil stress-strain and strength characteristics from the SBP [Schmertmann (1975)].

Although several aspects are important for a reliable determination of $\sigma_{ho}$, the following appear to have priority:

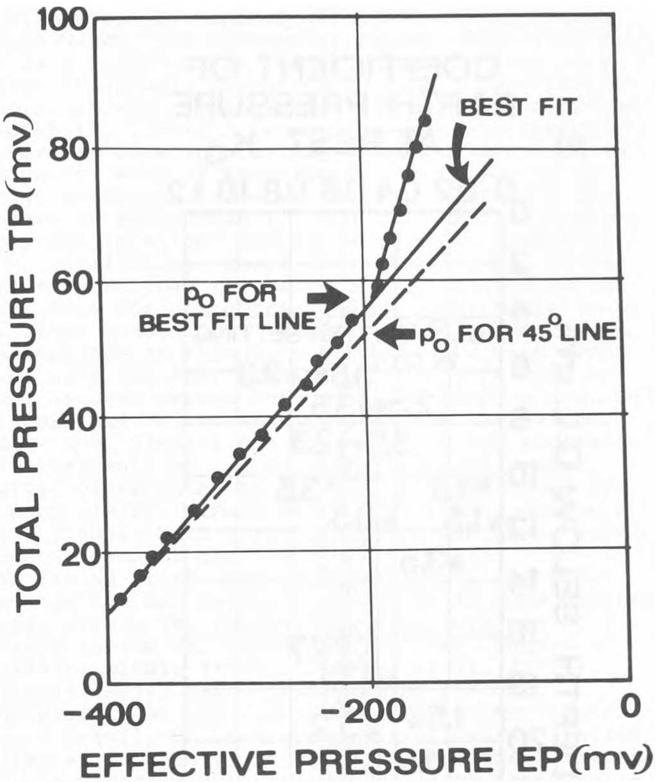



Fig.44: $p_o$ Determined from Total vs. Effective Pressure Plot [Wroth (1982),Lacasse and Lunne (1982)].

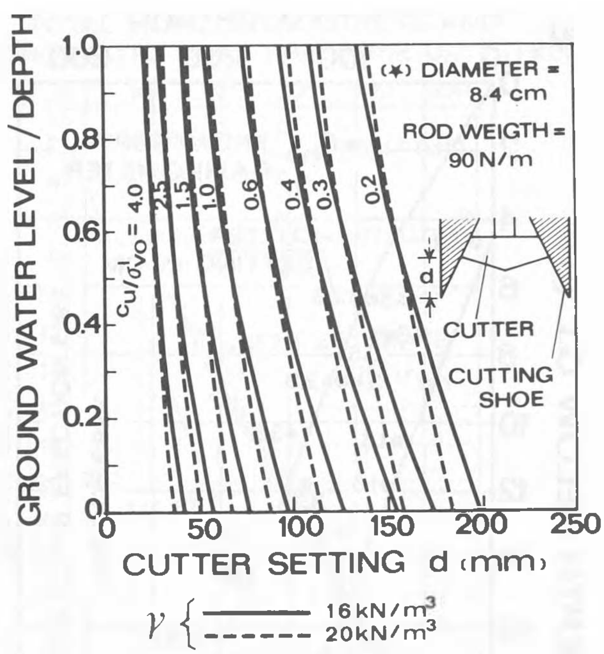



Fig.45: Cutter Setting for Clays, Applicable to Camkometer Type MK III* [Adapted from Clarke (1981)].

1. The reduction and if possible the elimination of the compliance of the strain feeler arms of the Camkometer as well as their careful calibration [see Dalton and Hawkins (1982), Benoit (1983)].

2. The insertion of probes with a perfectly cylindrical shape,a condition that is easier to reach when using a probe with an internal rigid body [see Law and Eden (1980, 1982), Fahey and Randolph (1983), and Benoit (1983)].

3. The cutter position with respect to the cutting shoe edge. This problem has been theoretically examined for clay by Clarke (1981) and subjected to field experiments by Hughes (1973), Windle (1976),Clarke (1981) and Steussy (1980). Clarke(1981) presented a formula relating the cutter setting to the equipment characteristics and undrained clay strength. The application of this formula to the MK-3 version of the Camkometer probe is shown in Fig.45.
There is a lack of experience regarding the appropriate cutter setting in sands. SBP tests recently performed in clean dense quartz sand by TUT, with a cutter setting of ≅ 2.3 to 3.3 cm and insertion rate of 2 to 3 cm/min, led to apparently reasonable results (see Fig.46).

4. The geometrical configuration of the cutter and its relation to soil type [see Clarke(1981), Fahey and Randolph (1983)].

5. The drilling rig characteristics,particularly vibrations and eccentric movements which may be transmitted through the rods to the probe.
In this respect, the provision for a rotary head at the top of the PAFSOR probe seems to be particularly successful.
With the Camkometer, which is drilled into the soil by a rotary head operating from ground level, recent experience at TUT seems to indicate that the use of a single instead of a double hydraulic piston rig leads to an improvement in the quality of probe penetration.

6. The characteristics of the self-boring process, including penetration rate, drilling mud pressure and flow rate, rate of motor rotation, etc.[see Hughes (1973), Denby (1978), Windle (1976), Clarke (1981), Ghionna et al. (1982a) and Benoit (1983)].All of these features and the cutter setting are interdependent. In the case of clays, they are generally selected in such a way as to ensure that the pore pressure measured during probe insertion should remain as close as possible to the existing value of $u_o$.

7. The relaxation time [Ghionna et al. (1981, 1983), Lacasse and Lunne (1982)] which is the waiting time which must elapse between probe insertion and the beginning of the expansion test. The intention is that during this waiting period, the external stress state around the probe, always disturbed to some extent,even by the self-boring process, will reach equilibrium. This condition is monitored by measuring the variation of the total external stress (PAFSOR) and/or the pore pressure (Camkometer) with time.
Finally, it must be pointed out that a reliable measurement of $\sigma_{ho}$ from the initial part of the expansion curve requires a sufficient number of readings before "lift-off" occurs. This requires a careful selection of the expansion or loading rates at the beginning of the test. Clarke (1981) recommends that the initial stage of the expan-

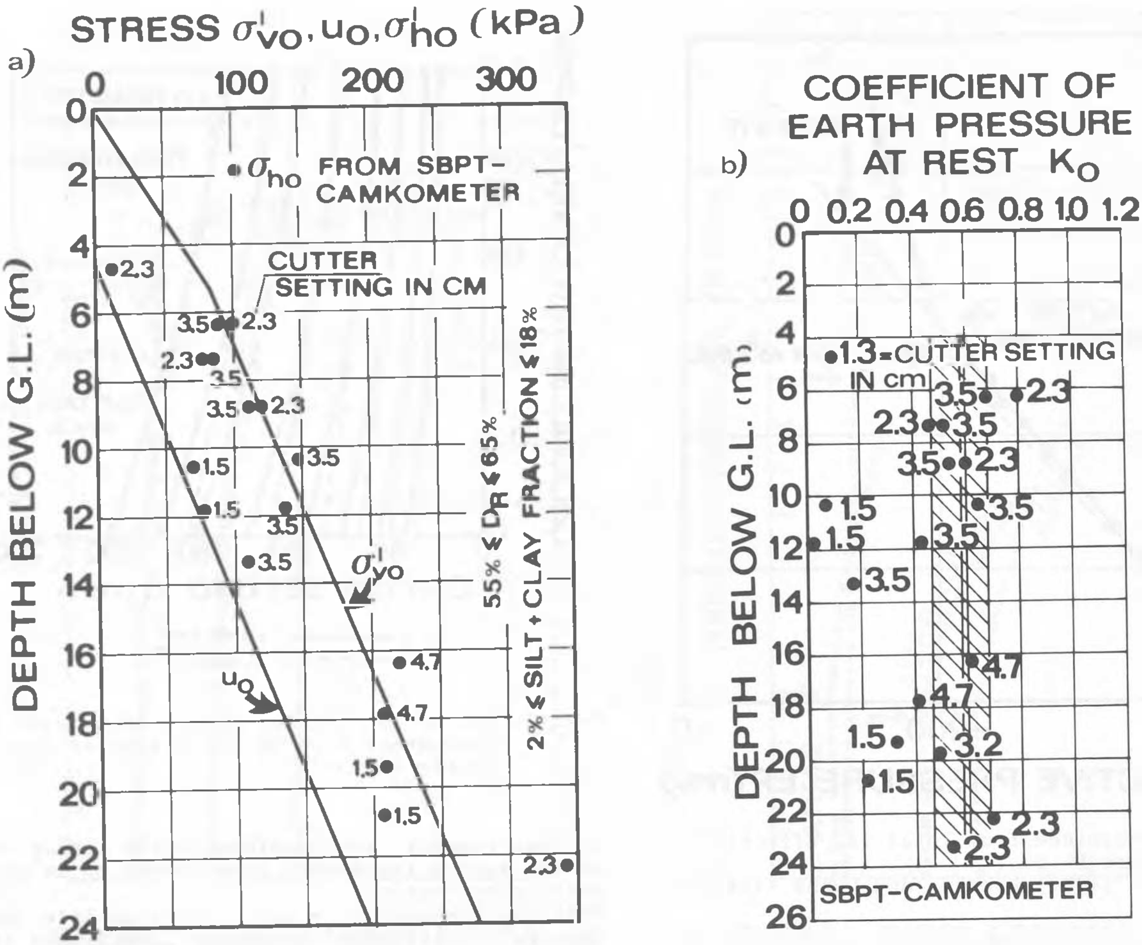



Fig.46: Influence of Cutter Setting, (a) on Measured $\sigma_{ho}$ and (b) on $K_o$, in a Medium to Dense Slightly Silty Sand from the Po River Valley [data from Ghionna (1984)].

sion be performed at a constant loading rate until the lift-off pressure is reached.
Apart from this requirement, the measured $\sigma_{ho}$ does not appear to be affected by the expansion rates used [Benoit (1983)].

Despite all this accumulated experience, the writers believe that it may still be difficult to select "a priori" the most appropriate insertion and expansion procedure for a site. Therefore, if the SBP tests have to be carried out at new sites, it is highly advisable to perform some preliminary tests to optimize the self-boring process.
During the last ten years, extensive experience has been gained in the measurement of $\sigma_{ho}$ in cohesive deposits.
Fig.47 shows the results of measurements of $\sigma_{ho}$ with the Camkometer in a soft, highly plastic, slightly organic clay (OCR ≅ 1.1 ± 0.1) encountered in the bay at Panigaglia (La Spezia), Italy. The SBP $\sigma_{ho}$ is compared with the best estimate of $\sigma_{ho}$ obtained from laboratory tests, and the overall agreement between the two measurements is good.
Fig.47 shows that the three strain arms considered separately yield slightly different values of $\sigma_{ho}$, a problem which has been discussed previously.

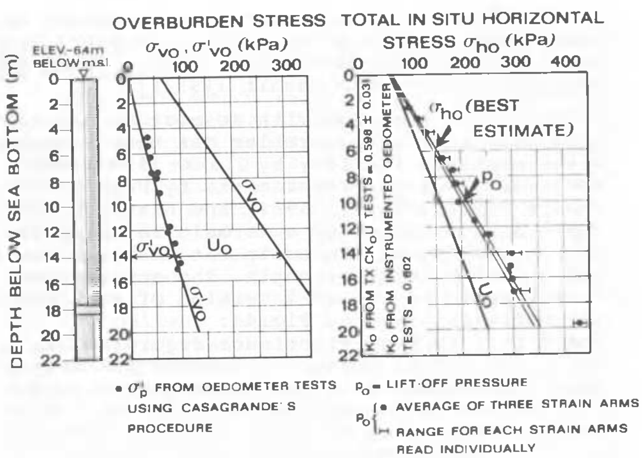



Fig. 47: Total In Situ Horizontal Stress in Very Soft Silty Clay at the Panigaglia Site as Measured During SBP Tests.

Fig.48 shows values of $\sigma_{ho}$ measured at the Porto Tolle site [Ghionna et al. (1981, 1983)] using both PAFSOR and Camkometer probes. This site with a 23 m thick recent deposit of soft silty clay is relatively well documented from a geotechnical point of view [Jamiolkowski et al. (1980), Ghionna et al. (1981)]. As mentioned in section 3.2.9. there is some uncertainty about its stress history However, even if one accepts the values of $\sigma'_p$ from oedometer tests, the OCR of this soil appears to be between 1.1 and 1.3 (Fig.60 ), probably due to aging combined with a few meters of GWL oscillation. Under these conditions, it may be assumed that the value of $\sigma_{ho}$ in situ is not far from the one corresponding to normally consolidated conditions. Therefore, the values of $\sigma_{ho}$ determined at this site from the SBP were compared with the "best estimate" of the in situ total lateral stress deduced from other in situ and laboratory tests, assuming for the latter that $K_o = K_o^{NC}$. Similar and apparently quite successful measurements of $\sigma_{ho}$ in other soft and medium stiff clays with the SBP has been described by Denby (1978), Clough and Denby (1980), Hughes et al. (1980), Mori (1981), Lacasse and Lunne (1982), and Ghionna et al. (1981, 1982, 1983).

As far as stiff clays are concerned, the experience for assessing $\sigma_{ho}$ from SBP is more limited and, with a few exceptions, restricted to OC clays in the UK, mainly London clay [Hughes (1973), Windle (1976), Simpson et al. (1979), Clarke(1981), Dalton and Hawkins (1982)].

An example of $\sigma_{ho}$ values from Camkometer tests in a heavily OC, cemented, microfissured Italian clay at Taranto is shown in Fig.49. These values are compared with those derived from the best estimate of $\sigma_{ho}$ from laboratory tests. They are based on:

- $u_o$ values from piezometer readings;
- OCR values from oedometer tests;
- parameters of the empirical relationship $K_o^{RB} = K_o^{NC} \cdot (OCR)^{\alpha}$ obtained from oedometer tests in which $\sigma_h$ is measured. In this expres-

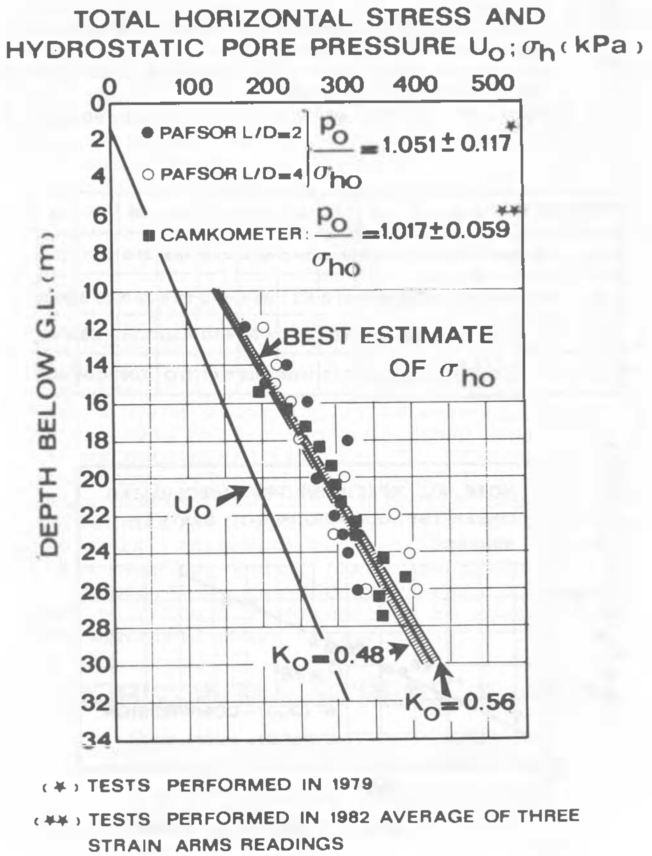



Fig.48: Total Horizontal Stress as Obtained from SBPT at the Porto Tolle Site [Ghionna et al. (1981, 1983)].

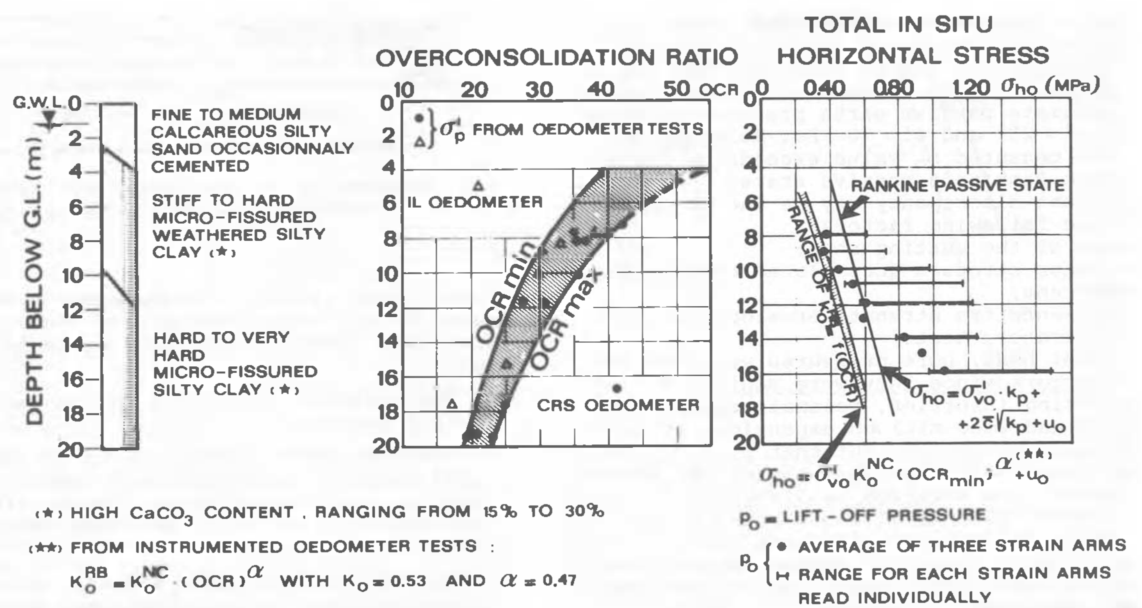



Fig.49: Total In Situ Horizontal Stress at the Taranto Site as Measured During SBP Tests.

sion:

$K_o^{RB}$ = coefficient of earth pressure during one-dimensional rebound
$K_o^{NC}$ = as above, but under NC loading conditions
$\alpha$ = experimental exponent

the effective stress-strength envelope shown in Fig.50.

| SPECIMEN N° | | 1 | 2 | 3 | 4 | 5 | 6 | 7 | 8 | 9 | 10 | 11 | 12 |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| $q_f$ | kPa | 28 7 | 34 9 | 23 4 | 33 6 | 36 0 | 39 5 | 49 4 | 27 6 | 49 9 | 21 0 | 35 1 | 33 7 |
| $p'_f$ | kPa | 47 6 | 58 7 | 40 6 | 69 4 | 57 2 | 65 1 | 89 4 | 42 8 | 95 8 | 32 0 | 60 8 | 60 3 |
| $\epsilon_f$ | % | 3.62 | 5.43 | 2.14 | 5.50 | 3.56 | 3.72 | 4.89 | 4.50 | 6.90 | 4.76 | 4.06 | 6.05 |
| $A_f$ | – | 0.07 | 0.03 | 0.03 | 0.10 | 0.12 | 0.06 | 0.03 | 0.01 | 0.12 | 0.09 | -0.04 | -0.05 |

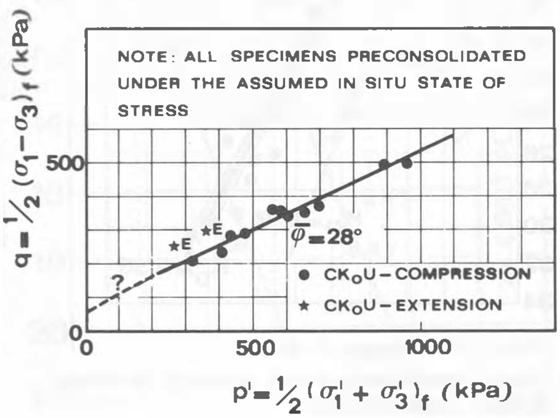



Fig.50: Effective Stress Envelope for Taranto Clay as Obtained from $CK_oU$ Triaxial Compression Tests.

Referring to the lift-off pressure which corresponds to the average of the three strain feeler readings, it may be observed that the $p_o$ values obtained above a depth of 12 m are close to the ultimate passive earth pressure calculated with $\phi' = 28°$ and c' = 70 kPa. Below this depth, the measured $p_o$ value exceeds $\sigma_h$ corresponding to Rankine's passive state.
This apparent discrepancy may be due to one or more of the following factors:
- clogging of the cutting shoe;
- large shear stresses accumulated between clay and membrane;
- nonrepresentative strength envelope obtained from laboratory tests.

On the other hand, pore pressures were not the cause of errors since they were monitored continuously during insertion, relaxation (ranging between 80 and 2000 min) and expansion. In addition, it must be pointed out that $p_o = \sigma_{ho}$ as obtained from the curve representing the average of the strain arm readings is always very close to the lowest value of lift-off pressure measured by the three feelers. Fig.49 shows that on the Taranto site very large differences between the $p_o$ values from individual strain feeler arms were obtained [see also Fig.43(b)].
Only limited experience exists in the evaluation of $\sigma_{ho}$ from SBP tests carried out in sands. Some experimental data and related comments may be found in Windle (1976), Fahey (1980), Wroth (1982, 1984), Fahey and Randolph (1983) and Mair and Wood (1985).
With sands, the importance of taking extreme care during probe insertion is self-evident; even a small disturbance leads to a situation in which the lift-off pressure matches closely the equilibrium pore pressure [see Windle (1976) and Fahey and Randolph (1983)]. The correction techniques described by Fahey and Randolph (1983) for obtaining a reliable estimate of $\sigma_{ho}$ from SBP tests not perfectly inserted seem to contain a lot of uncertainties.
In addition, in sands it is even more difficult than in clays to compare the $\sigma_{ho}$ values obtained from SBP with the horizontal in situ pressures determined by other means; in fact, the writers are not aware of any laboratory or in situ tests which offer a sufficiently reliable assessment of the total in situ lateral stress in cohesionless deposits.
During the preparation of this paper, TUT was conducting a series of SBP tests in a NC dense sand deposit in the valley of the River Po.
An example of raw data obtained from one of these tests in shown in Fig.51.

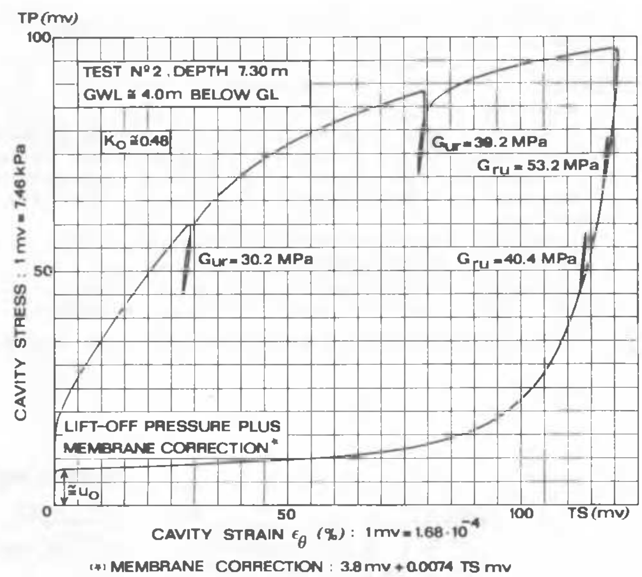



Fig.51: Example of SBP Camkometer Test in NC Sand in the Po River Valley (Raw Data).

These tests yielded an apparently reasonable range of $\sigma_{ho}$ and $K_o$ values, as shown in Fig.46. From this figure, the following comments can be made:

1. The measured values are influenced by the cutter setting.
2. A cutter setting of $\leq$ 1.5 cm in the tested sand leads to unsatisfactory insertions, so that the corresponding $\sigma_{ho}$ values are close the hydrostatic line or in any case obviously too low.
3. Excluding these tests and the one with the cutter setting 3.5 cm which gave an unusually low $K_o$ = 0.19, all the other computed $K_o$ values fall within a range between 0.4 and 0.8.

The average value of all $K_o$ results equals 0.58±0.11.

This value of $K_o$ is too high for a deposit for which all available geological information indicates no possibility of mechanical overconsolidation. However, oedometer tests run on samples taken from cohesive lenses embedded in the sand deposit at depths ranging between 18 m and 70 m below existing G.L. indicate values of $\sigma'_p$ which are consistently above $\sigma'_{vo}$ leading to OCR values between 1.3 and 1.6.
The preconsolidation mechanism is very difficult to assess in the examined case. The writers are inclined to attribute it to aging and/or cementation effects combined with modest G.W.L. oscillations.
The expected "reasonable" range of $K_o$ values may estimated to be between 0.4 and 0.5.

### 3.2.5. $\sigma_{ho}$ from Push-In Total Stess Cells (TSC)

Since the late sixties, attempts were made to measure $\sigma_{ho}$ by pushing total stress cells (TSC) vertically into natural soil deposits. It was assumed that once the cell has been inserted and a sufficiently long waiting period had elapsed, the excess pore pressure caused by the installation would dissipate and the total stress on the face of a perfectly vertical TSC would come to equilibrium and be close to $\sigma_{ho}$. Unfortunately, this approach for the evaluation of $\sigma_{ho}$ is somewhat limited in other than soft clays having low OCR.

The main problems are:

- It has not been demonstrated conclusively, even with the installation of an almost infinitely thin cell and after a sufficiently long relaxation period, that the soil stresses will return to the values which existed prior to insertion. Also, under these idealized conditions, there remains the fact that now a rigid inclusion exists in the soil.
- A more realistic assumption is that the push-in TSC, even with cell thicknesses as small as 0.3 to 0.5 cm, leads to a bedding error, even after a very long relaxation period. This in turn will lead to an overestimate of $\sigma_{ho}$, especially in highly OC clays.
  The bedding error depends on numerous factors like the geometry of the cell, its overall stiffness with respect to that of the soil in which it is embedded, the characteristics of the flexible membrane through which the soil pressure is measured, etc.; for details see Hanna (1985).

In the seventies, a hydraulically operated TSC was developed which permitted apparently successful measurements of $\sigma_{ho}$ to be made in soft clays. Measurements of this type have been well-documented by Massarsch (1975, 1979), Massarsch et al. (1975), and Tavenas et al. (1975).
The application of the technique to stiff clays has been tried only recently by Tedd and Charles (1981, 1983) and by Powell et al. (1983). Tedd and Charles (1981, 1983) presented a comprehensive record of $\sigma_{ho}$ measurements in stiff London clay using 10 cm wide, 20 cm long and 5 mm thick spade-like cells which were hydraulically pushed into the soil. These cells were installed both vertically from G.L. and horizontally from an existing cut. The horizontal cells which, in principle, measured the existing and easily determined total overburden stress enabled an assessment of the bedding error to be made, which in the case of push-in cells, results in too large readings. On the basis of their own experimental data, other data available from literature, and on a simplified elastic analysis, Tedd and Charles (1983) concluded that these over-readings should be proportional to the stiffness of the clay.
Due to the fact that in clays one generally relates the stiffness empirically to the undrained shear strength, and also because $c_u$ is more easily available than Eu, Tedd and Charles (1983) present a correlation between $c_u$ and the over-reading of the push-in TSC; it is obviously valid for cells with the same geometrical dimensions as the ones used by them. From the correlation presented by Tedd and Charles (1983), it appears that in soft clays with $c_u \leq 30$ kPa, the over-reading is negligible. For higher undrained strengths ($c_u > 30$ kPa) the push-in cells tend to over-read $\sigma_{ho}$ by 40 to 50% of the value of $c_u$ (undrained shear strength obtained from UU triaxial compression tests).
Fig.52 shows a comparison between $\sigma_{ho}$ measured in London clay by means of SBP-Camkometer tests and values determined with the TSC. The same figure also shows values of $\sigma_{ho}$ deduced from laboratory measurements of capillary pressure [Skempton (1961) and Burland and Maswoswe (1982)]. On the average there is agreement between SBP and laboratory tests results, and if these are taken as reference values, then the TSC over-reads $\sigma_{ho}$ by ≅ 60 kPa; this roughly corresponds to about 50% of the undrained shear strength.

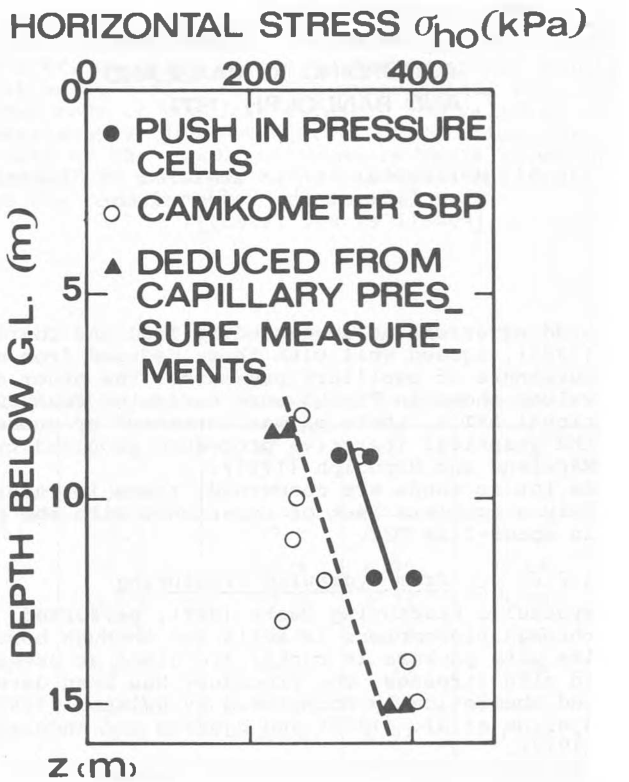



Fig.52: Horizontal Stress Measured in London Clay at the Essex Site [Tedd and Charles (1983)].

Powell et al. (1983) reported a series of apparently successful measurements of $\sigma_{ho}$ with a push-in TSC in stiff glacial till with the same type of instrument as used by Tedd and Charles (Fig.53). The measured $\sigma_{ho}$ values, corrected for

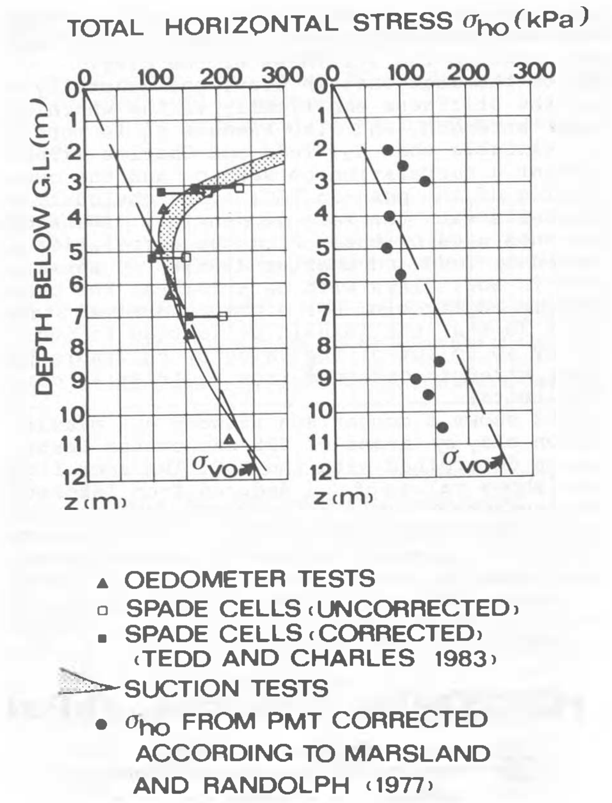



Fig.53: Horizontal Stress Measured in Glacial Clay Till at the Cowden Site [Powell et al. (1983)].

bedding errors as suggested by Tedd and Charles (1983), agreed well with those deduced from measurements of capillary pressures. The other $\sigma_{ho}$ values shown in Fig.53 were estimated from conventional PMT's, where $p_o$ was corrected by means of the graphical iterative procedure proposed by Marsland and Randolph (1977).
As far as sands are concerned, there is unfortunately a complete lack of experience with the push-in spade-like TSC.

### 3.2.6. $\sigma_{ho}$ from Hydraulic Fracturing

Hydraulic Fracturing Tests (HFT), performed through piezometers in soils and through boreholes with packers in rocks, are aimed at assessing in situ stresses. The procedure has been developed and theoretically formulated by Haimson (1968), Bjerrum et al. (1972) and Bjerrum and Anderson (1972).
Basically, the HFT consists of gradually increasing the water pressure in a piezometer and monitoring the water outflow rate for 3 to 10 minutes at each pressure step. Proceeding in this manner, a pressure is reached at which a large increase in the flow rate occurs and this means that a crack has formed in the soil.
The pressure is then reduced incrementally until it drops to precracking values, while the flowrate is continuously monitored. At this point, the pressure is increased again until it just reaches the cracking pressure. This operation is repeated a few times in order to determine alternatively the flow rates during and after cracking.
The underlying principle is that the hydraulic fracture will propagate in a direction normal to that of the minor principal stress, $\sigma_3$. This poses an obvious limitation on the HFT in that $K_o$ must be less than 1.
The interpretation of the test is subject to some inherent uncertainties [Marr (1974)]:

- The mechanism of the cracking itself may be of different types; fracturing may occur under steady flow conditions (penetrating) or under almost undrained conditions (no penetration).
- There is some (unknown) influence of the piezometer installation procedure on the stress conditions in the surrounding soil mass, and consequently, on the pressure causing hydraulic cracking [see also Bjerrum and Anderson (1972)].
- The soil tensile strength has an influence which may be different under drained and undrained conditions [see also Haimson (1968)].
- The soil macrofabric and, generally speaking, any inhomogeneities in the soil will influence the test results.

The existing experimental evidence, which is at least partially supported by theory [Marr (1974), Bozozuk (1974), Massarsch et al. (1975), Tavenas et al. (1975), Penman and Charles (1981)], permits the following conclusions:

- The hydraulic fracturing pressure is influenced mainly by the minor principal stress and the tensile strength of the tested soil.
- There is disagreement about the influence of the piezometer installation method [Marr(1974), Tavenas et al. (1975)].
- The close-up pressure measured at the end of the test, when tensile strength effects have been presumably cancelled, yields values of $\sigma_3$ and, hence, in soils with $K_p < 1$, allows the evaluation of $\sigma_{ho}$.

### 3.2.7. $\sigma_{ho}$ from Marchetti Flat Dilatomer (DMT)

The evaluation of the initial lateral stress existing in the ground prior to the insertion of the dilatometer is based on the idea [Marchetti (1980)] that the lateral stress index

$$K_D = \frac{p_o - u_o}{\sigma'_{vo}}$$

which represents effective stress ratio, may be correlated to the coefficient of earth pressure at rest, $K_o$. The above symbols denote:
$p_o$ = first corrected dilatometer reading
$u_o$ = equilibrium pore water pressure
$\sigma'_{vo}$ = effective overburden stress
On the basis of data collected in Italy from a number of sites with hard-mineral sand and soft to medium stiff clays of low sensitivity, Marchetti (1980) proposed the following tentative correlation between $K_o$ and $K_D$, valid for $K_o \geq 0.3$:

$$K_o = \left(\frac{K_D}{2}\right)^{0.47} - 0.6$$

He defined a precise limit for the validity of this empirical formula, restricting its application to materials not subjected to cementation, etc. and which, if mechanically OC, have experienced only a simple unloading. The validity of this relationship has been mainly documentated for cohesive deposits [Marchetti (1980), Lacasse and Lunne (1982b), Campanella and Robertson (1983)].

As far as cohesionless soils are concerned, the accumulated experience with uncemented hard-mineral deposits shows [Schmertmann (1983), Marchetti (1982), Campanella and Robertson (1983)] that the same equation is not realistic. In fact, both field tests and laboratory tests in the calibration chamber on artificially deposited sand [Jamiolkowski et al. (1979), Marchetti (1982)] have led to the conclusion that $K_D$ is mainly controlled by two factors, $D_R$ and $\sigma_{ho}$. Therefore, in order to evaluate in a reliable way $K_o = f\ (K_D)$, it is necessary to separate the influence of $D_R$ on $K_D$ from that of $\sigma_{ho}$.

Based on the results of a limited number of calibration chamber tests run at the University of Florida and by ENEL CRIS in Italy, Schmertmann (1983) proposed a new tentative procedure for evaluating $K_o = f\ (K_D)$. The procedure is applicable to soils in which the DMT is run under virtually drained conditions ($I_D$ is higher than 1.2). It consists of the following steps:

1. Measure $K_D$.
2. Determine $q_D$ or $q_c$ at the same depth.
   $q_D$ = bearing resistance measured at the tip of the dilatometer during penetration
   $q_c$ = cone resistance from CPT.
3. Assume a trial value of $K_o$.
4. Estimate a trial value of $\bar{\phi}_{PS} = f\ (q_D)$ or $\bar{\phi}_{AX} = f\ (q_c)$ according to Durgunoglu and Mitchell (1973, 1975); see also Schmertmann (1982)
   $\bar{\phi}_{PS}$ = angle of shearing resistance under plane strain conditions, derived as function of $q_D$ by Durgunoglu and Mitchell's theory (1973); see also Schmertmann (1982)
   $\bar{\phi}_{AX}$ = angle of shearing resistance under conditions of axial strain symmetry, derived as a function of $q_c$ according to Durgunoglu and Mitchell (1973)
5. If $\bar{\phi}_{PS}$ is estimated from $q_D$, convert it to $\bar{\phi}_{AX}$ by one of the following approximate formulae:

   $$\bar{\phi}_{AX} \cong \bar{\phi}_{PS} - \frac{1}{3}\ (\phi_{PS} - 32) \quad \text{for } \phi_{PS} > 32°$$

   $$\bar{\phi}_{PS} = \bar{\phi}_{AX} \quad \text{for } \phi_{PS} \leq 32°$$

   If $\phi_{AX}$ has been deduced from $q_c$, proceed directly with the next step.
6. Compute $K_o$ from the following equation:

   $$K_o = \frac{1}{192 - 717\ (1 - \sin \bar{\phi}_{AX})} \cdot \left[40 + 23\ K_D - 86\ K_D\ (1 - \sin \bar{\phi}_{AX}) + 152\ (1 - \sin \phi_{AX}) - 717\ (1 - \sin \phi_{AX})^2\right]$$

7. Compare this value of $K_o$ with the trial value assumed at step No.3.
8. Iterate from step No.2 until the values of $K_o$ agree to within ± 10%.

In order to check the reliability of this procedure, the writers have examined six calibration chamber tests carried out on pluvially deposited medium fine quartz sand from the Ticino River [Jamiolkowski et al. (1979), Baldi et al. (1981, 1982, 1985), Bellotti et al. (1982)]. In these tests, $K_D$ was measured on one-dimensionally consolidated sand specimens 1.5 m in height and 1.2 m diameter, of which both $q_c$ and the applied boundary stresses were known. The experimental data obtained made it possible to check the validity of Schmertmann's (1983) tentative correlation. The results of the check are shown in Table IX which compares the measured $K_o$ to the value obtained from the formula ($K_o^{SCH}$) in step No.6.

TABLE IX

Comparison Between $K_o$ of Ticino Sand Predicted from DMT and $K_o$ from Calibration Chamber Tests

| Test No. | $\gamma_D$ kN/m³ | $D_R$ % | OCR - | $\sigma'_{vp}$ kPa | $K_o$ - | $q_c$ MPa | $\phi'_{DM}$ ° | $K_D^*$ - | $K_o^{SCH}$ - |
|---|---|---|---|---|---|---|---|---|---|
| 41 | 16.19 | 66.7 | 1 | 117 | 0.581 | 9.4 | 37.8 | 6.11 | 0.84 |
| 42 | 16.23 | 77.6 | 1 | 321 | 0.510 | 24.2 | 37.6 | 4.55 | 0.66 |
| 43 | 16.22 | 77.3 | 1 | 118 | 0.400 | 13.4 | 39.8 | 4.24 | 0.48 |
| 44 | 16.29 | 79.3 | 1 | 221 | 0.414 | 20.4 | 38.8 | 4.36 | 0.57 |
| 46 | 16.22 | 77.3 | 2.73 | 118 | 0.727 | 15.0 | 39.6 | 7.39 | 0.88 |
| 47 | 16.23 | 77.6 | 1 | 66 | 0.390 | 9.9 | 41.1 | 4.33 | 0.37 |
| 49 | 16.30 | 79.6 | 5.41 | 116 | 0.963 | 19.4 | 40.4 | 8.93 | 1.00 |

$\sigma'_{vp}$ = effective vertical stress during penetration

$K_o$ = coefficient of lateral stress at mid-height of the CC specimen during the DMT

$\phi'_{DM}$ = angle of shearing resistance as a function of $K_o$ and $q_c$ according to Durgunoglu and Mitchell (1973)

$K_D$ = lateral stress index from DMT

* = all DMT's were run under almost constant boundary stress conditions, with the exception of tests No.41 and No.42 which were carried out with $\sigma'_{vp}$ = constant and $\varepsilon_h = 0$

Considering the ideal laboratory conditions and the artificially deposited sands, it appears that Schmertmann's (1983) correlation is able to predict within a reasonable degree of accuracy the value of $K_o$. On the average, there is an overall tendency for $K_o$ to be overestimated, and the ratio of Schmertmann's $K_o$ to the actual is 1.22 ± ± 0.18.

A second application of the above procedure for the evaluation of $K_o = f(K_D)$ was under field conditions, in the recent deposit of sand already mentioned in the discussion of the SBP tests performed in the Po River Valley (section 3.2.4). At this site investigations are currently being carried out with a series of CPT, DMT, and SBP tests. Fig. 54 reports $K_o = f(K_D)$ values obtained by Schmertmann's (1983) approach, in which $\bar{\phi}_{AX}$ was evaluated on the basis of $q_c$ measurements with the standard electrical friction cone at a distance of ≅ 3 m from where the DMT's had been performed.

For comparison, it may be mentioned that a preliminary interpretation [Ghionna (1984)] of SBP - Camkometer tests at the same place at depths ranging from 6 to 20 m led to $K_o$ values varying between 0.40 and 0.80. An example of the SBP results was shown in Fig.46(b).

As mentioned in Section 3.2.4, the deposit is either NC or slightly OC with the preconsolidation mechanism due to aging and GWL oscillations. From Fig.54, it appears that also under field conditions, the equation suggested by Schmertmann (1983) for estimating $K_o = f(K_D)$ of sands yields reasonable results which, on the average, fall within the range of the values obtained from the SBP.

The situation would probably be improved by:

- carrying out a large number of additional calibration chamber tests on different sands at a wide range of OCR;
- incorporating in the interpretation of the calibration chamber tests the effects of boundary conditions and chamber size [Parkin and Lunne (1982), Baldi et al. (1982), Bellotti et al. (1983)];
- an analysis taking into consideration the fact that the measured $q_c$ reflects implicitly both the nonlinearity of the shear strength envelope [Baligh (1975), Bellotti et al. (1983)], and the development of progressive failure around the cone; both phenomena lead to an underestimate of $\phi'$ when using Durgunoglu and Mitchell's theory (1973, 1975);
- developing an improved theory for the evaluation of $\phi'$ on the basis of the penetration resistance, since the one presently used incorporates many simplifying assumptions; the most important assumption appears to be an underestimate of the influence of $\sigma'_{ho}$ on the computed values of $q_c$ and $q_D$.

In spite of its largely empirical nature, the Schmertmann (1983) procedure, using the lateral stress index $K_D$, allows a reasonable estimate of $K_o$ in sands to be made

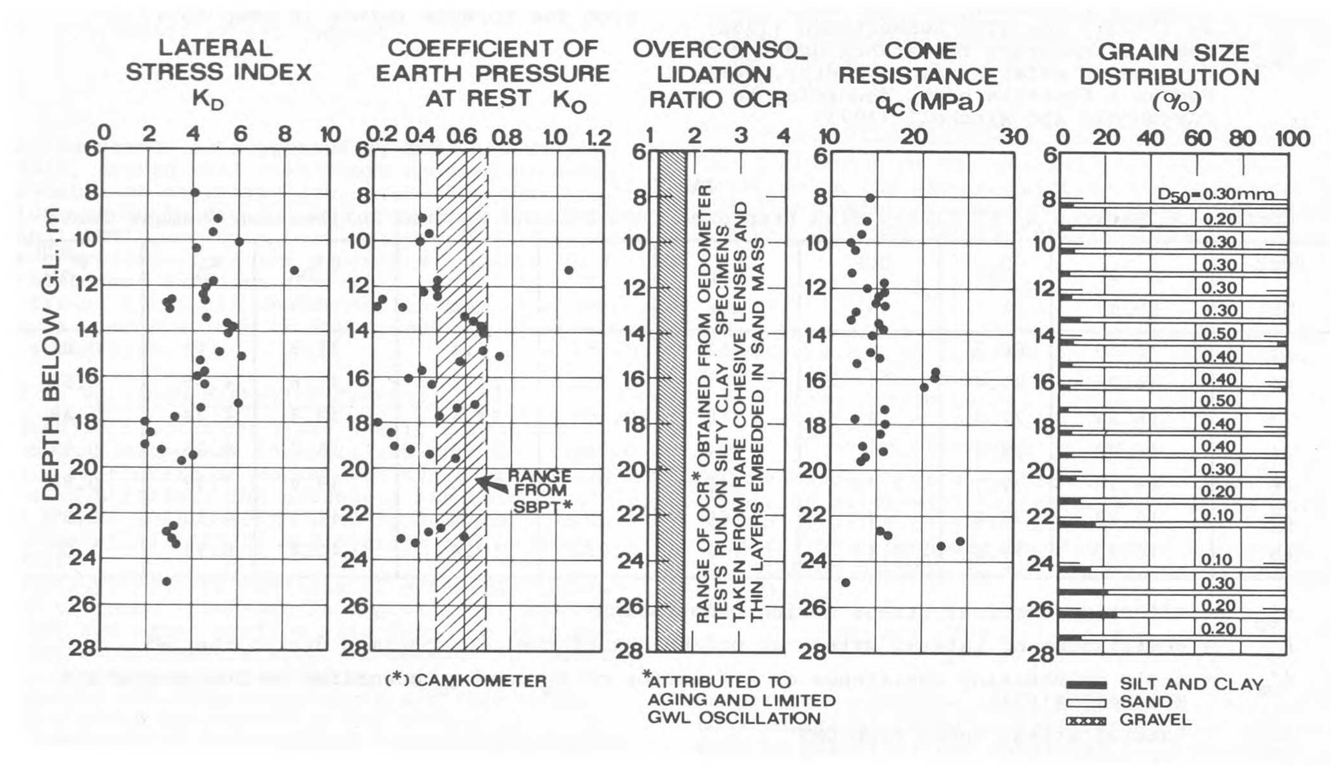



Fig.54: Evaluation of $K_o$ in Sand from DMT According to Schmertmann (1983).

3.2.8. $\sigma_{ho}$ from the Iowa Stepped Blade (ISB)

The recently developed (ISB) [Handy et al. (1981, (1982)] shown in Fig.55, consists of a series of three or four passive, pneumatic, total stress cells of variable thickness.

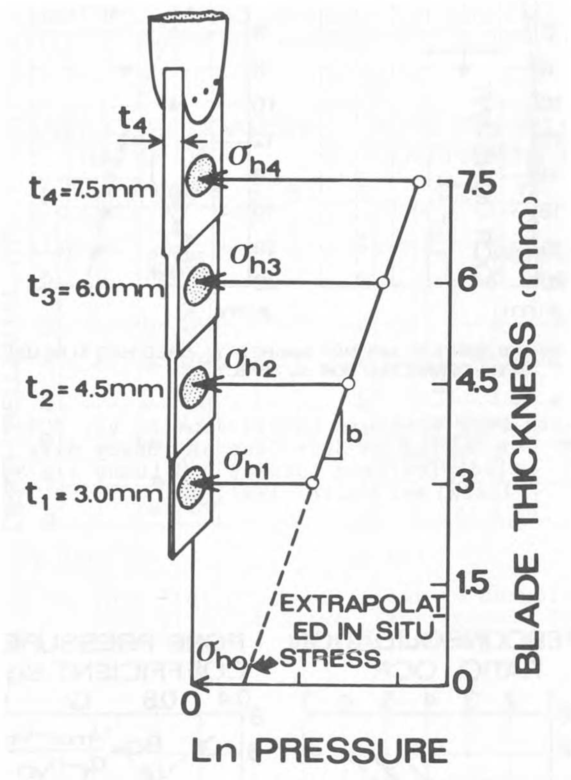



Fig.55: Iowa Stepped Blade

This device is an extension of the spade-like TSC instrument, with the difference that, according to Handy et al. (1982), it is not necessary in this case to wait for the equilibrium pressure to be established in order to evaluate $\sigma_{ho}$.
The principle underlying the ISB tests is opposite to that of the SBP. It is assumed that the disturbance caused by the insertion of any device into the soil is unavoidable and that it varies as a function of the thickness of the device. It is therefore postulated that, after probe insertion, the lateral pressures $\sigma_{h1} < \sigma_{h2} < \sigma_{h3} < \sigma_{h4}$ are measured and the values correspond to the respective thicknesses $t_1 < t_2 < t_3 < t_4$ of the total stress cells. Thus the $\sigma_h$ value obtained can be extrapolated in the direction of a thickness equal to zero, and this should be the total lateral stress, $\sigma_{ho}$, for the undisturbed state (Fig.55). This working hypothesis has been documented by a limited number of laboratory and in situ tests which allowed Handy et al. (1981, 1982) to postulate tentatively the following exponential relationship between $\sigma_h$ and t:

$$\sigma_h = \sigma_{ho}\ a \cdot e^{bt}$$

where a and b are regression coefficients and t = blade thickness.
At present, because of the limited experience with the ISB, Handy et al. (1981, 1982) recommend using a = 1 which means the total lateral in situ stress is $\sigma_h$ for t = 0. It must, however, be realized that this assumption implies no bedding errors for blades of zero thickness.
Experience with the ISB in a number of soils indicates that the values of b are generally between 0.12 $mm^{-1}$ for soft soil and 0.48 $mm^{-1}$ for hard and dense soils. According to the developers the adopted test procedure permits a very good duplication of the experimental data. The data reported by Handy et al. (1981, 1982) show that the measured pressures, $\sigma_h$, do not always steadily increase to higher values as blade thickness increases. This was noticed quite frequently for the thickest blade. The phenomenon might be attributed to the fact that the insertion of a blade beyond a certain thickness stresses the soil up to the limiting pressure or causes other serious changes in soil behaviour. Handy et al. (1982) reported an example of $\sigma_{ho}$ values obtained from ISB tests in a stiff to hard expansive slickensided clay and compared them to results of $\sigma_{ho}$ from SBP and laboratory tests. Even if the characteristics of the tested deposit are complex, the results so far seem to confirm that ISB-derived $\sigma_{ho}$ values compare well with those obtained from the SBPT.
It must, however, be borne in mind that the ISB has only very recently been developed, and therefore it requires further calibration work, both in the field and the laboratory. This is particularly true if one considers that:

- the laboratory tests with the ISB have been carried out under conditions such that the results must be considered qualitative rather than quantitative;
- the field tests available to the writers have generally been performed in quite complex soil deposits.

Under such circumstances, the device should be tested in laboratory calibration chambers (under strictly controlled test and boundary conditions) and in the field (in relatively homogeneous and geotechnically well-known deposits). This research program should be directed towards:

- verification of the tentative assumption that a = 1 for t = 0, and that no bedding errors occur due to rigid inclusions in the soil;
- examination of the possible influence on the extrapolated value of $\sigma_{ho}$ of the time interval between blade penetration and the end of the measurement of $\sigma_h$ in clays;
- examination of the influence of possible deviations of the ISB from a vertical direction;
- incorporating a pore pressure sensor on one or more of the blade steps to monitor pore pressures during the tests.

3.2.9. Stress History from Piezocone (CPTU)

At present there is almost a complete lack of experience of the assessment of stress history (OCR or $\sigma'_p$) from in situ tests.
Baligh et al. (1978, 1980) and Baligh and Vivatrat (1979), on the basis of experience with the PPP and CPTU in Boston Blue clay, suggested that the ratio of the pore pressure measured during penetration vs. $q_c$ may reflect the stress history of cohesive soil deposits. The reasoning behind this idea may be found in work by Baligh et al. (1980).
Using the concepts of Critical State Soil Mechanics, Wroth (1984) presented a comprehensive and critical review of the same idea. This analy-

sis may summarized as follows:

1. Any pore pressure coefficient derived from CPTU that can be correlated to OCR should be analogous to that formulated by Henkel (1960):

$$a_f = \frac{\Delta u - \Delta\sigma_{oct}}{\Delta\tau_{oct}}$$

where:

$a_f$ = Henkel's pore pressure coefficient at failure

$\Delta\sigma_{oct}$ = change in total octahedral stress

$\Delta\tau_{oct}$ = change in octahedral shear stress

2. As the actual $\Delta\sigma_{oct}$ and $\Delta\tau_{oct}$ around the CPTU tip are not known, it is necessary to refer tentatively to the parameter

$$B_q = \frac{u_{max} - u_o}{q_c - \sigma_{vo}}$$

which reflects at least to some extent the (dimensionless) ratio of the measured $\Delta u$ to the induced shear stress.
The writers agree with Wroth (1984) that $B_q$ should reflect better the in situ OCR than $u/q_c$ or $\Delta u/q_c$.

As in all applications of the CPTU, the question arises as to the optimum position of the filter stone in terms of the objectives of the test. Wroth (1984) postulated that the best position of the porous stone would be 5 to 6 times the cone diameter behind the base of the cone, because at that location a greater proportion of the measured $\Delta u$ is due to the induced $\Delta\tau_{oct}$ as compared to that caused by $\Delta\sigma_{oct}$. In that location also, the gradients of $\Delta u$ with respect to the axial distance from the tip are smaller, which leads to better accuracy and repeatability of the excess pore pressure measurements.
On the other hand there are some doubts [e.g. Roy et al. (1981)] if this measured $\Delta u$ is really the most sensitive to the stress history of the deposit since the clay is completely remolded at that location.
The writers believe that the location of the porous stone at or near the cone base is well suited to reflect the changes in OCR, and it represents an acceptable compromise given our present understanding of the topic.
Existing experimental evidence of the response of the CPTU to the stress history of clay deposits may be found in Baligh and Vivatrat (1979), Baligh et al. (1978, 1980), Hajibokar (1981), Tumay et al. (1981), Roy et al. (1981), Lacasse et al. (1981), Lacasse and Lunne (1982a), Smits (1982), Battaglio and Maniscalco (1983). Some examples of the observed trends of $B_q$ vs OCR are shown in the following figures:

- Fig.56 gives the variation of $B_q$ vs depth as obtained by Lacasse and Lunne (1979, 1982a) in the soft to medium sensitive clay at the Onsöy site in Norway.
  In the upper part of the deposit where OCR ranges betwen 9 and 1.5, there is a well defined trend of $B_q$ to increase as OCR decreases with increasing depth.
  However below the depth of 5 m where OCR is almost constant (1.1 to 1.2) the $B_q$ still tends to increase slightly with the depth.

- Fig.57 shows the variation of $B_q$ vs. depth as observed in Boston Blue clay (adapted from data by Baligh et al., 1981). At that time the measurements of $u_{max}$ and $q_c$ were made separately and the data assume that $q_t \cong q_c + 0.2\ u_{max}$. A well defined trend of increasing $B_q$ with decreasing OCR is obtained.

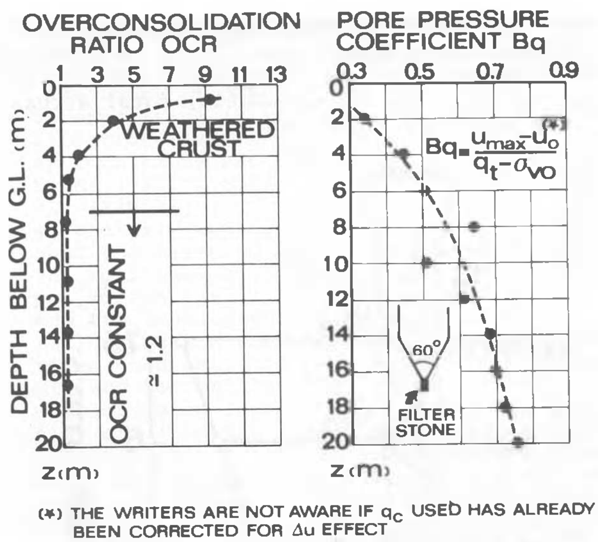



Fig.56: Pore Pressure Coefficient $B_q$ vs. OCR in a Medium to Soft Clay at Onsöy Site [Adapted from Lacasse and Lunne (1979, 1982a) and Wroth (1984)].

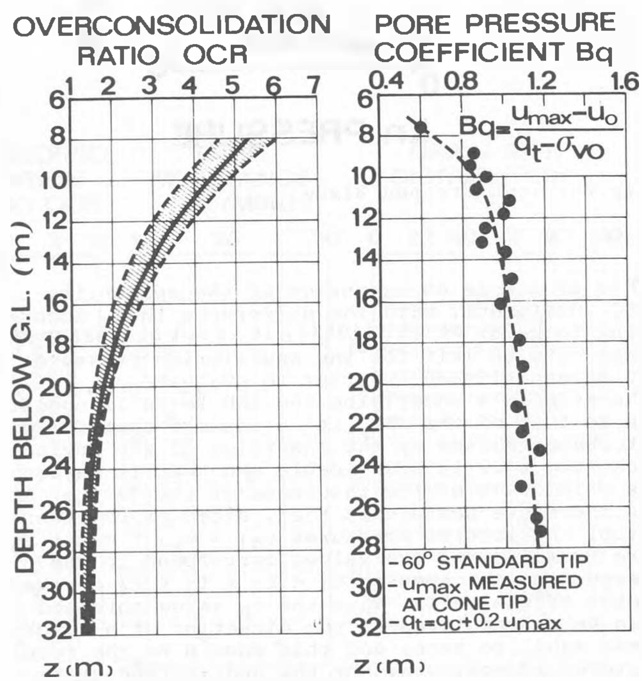



Fig.57: Pore Pressure Coefficient $B_q$ vs. OCR in Boston Blue Clay at Saugus Site, Mass. [Adapted from Baligh et al. (1981)].

- Fig.58 shows an interesting trend of $B_q$ vs depth for cemented sensitive Champlain clay at the St. Alban site in Canada. It was obtained from very comprehensive CPTU data by Roy et al. (1982, 1982a). In this case the OCR is almost constant with depth. The $B_q$ values determined for three different positions of the porous sensor are similar in shape and show only a slight tendency to increase with increasing depth.

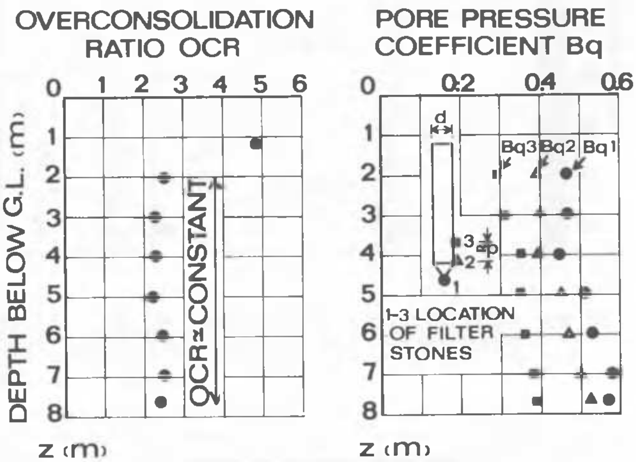



Fig.58: Pore Pressure Coefficient in Champlain Clay at Saint Alban [Experimental Data from Roy et al. (1982, 1982a)].

- Fig.59 gives the variation of $B_q$ with depth and with OCR as obtained in a silty clay deposit with well developed macro-fabric at the Pontida site in Italy [Battaglio and Bruzzi (1984)]. Also in this case a general tendency of $B_q$ to increase as OCR decreases may be observed. However, probably due to the macrofabric features of the deposit (thin seams and lenses of more permeable material embedded in the silty-clay mass), a large scatter of $B_q$ is observed.
At the depth between 18 and 20 m the $B_q$ values do not conform with the trend, probably due to a discontinuity in the OCR profile and/or to the presence of more pervious lenses.

- Fig. 60 reports the $B_q$ vs. depth observed at the Porto Tolle site in Italy. This is a very recent soft silty clay with a highly developed macro-fabric [see Battaglio et al. (1981)] and it had previously been considered to be a virtually NC deposit [see Bilotta and Viggiani (1975), Jamiolkowski et al. (1980), Ghionna et al. (1981), Battaglio et al. (1981), Jamiolkowski et al. (1982)]. Recently the writers reviewed the available oedometer and FV tests results and concluded that, despite the existing geological evidence, the in situ and laboratory tests yield $\sigma'_p$ = (1.1 to 1.3) $\sigma'_{vo}$. In this deposit the observed trend of $B_q$ versus depth is quite irregular and does not reflect the OCR profile. The writers do not know the reasons for this, but it probably is due to the presence of more pervious lenses and layers as shown in Fig.36.

- Finally Fig.61 shows the variation of $B_q$ obtained in heavily OC microfissured Taranto clay in Italy (see Fig.49). In this case the general trend of $B_q$ vs depth appears to be consistent

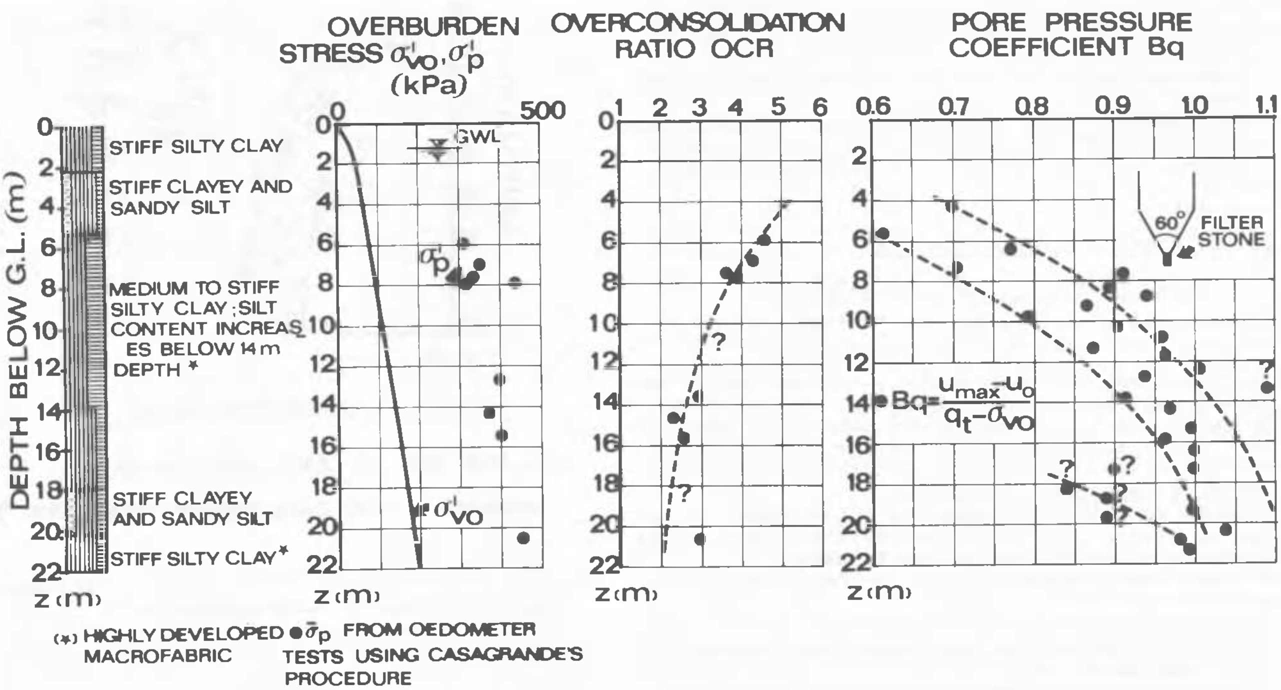



Fig.59: Pore Pressure Coefficient $B_q$ vs. OCR in a Medium to Stiff Silty Clay at the Pontida Site.

with the OCR profile even if there is significant scatter in the experimental data. This scatter reflects the scatter in OCR, which in turn may be due to a variation of the degree of cementation.
The higher $B_q$ values at the depth between 7 and 9 meters, may be attributed to the presence of a softened zone caused by a predrilled hole made a few days before the CPTU test.

This review of the potential of the CPTU to determine the stress history of cohesive deposits reveals the complexity of the topic and leads to the following preliminary remarks:

1. Based on theory and on soil behaviour observed during laboratory tests, the writers expect that $B_q$ reflects OCR changes within a given soil deposit. The examples of $B_q$ = f (OCR) shown in Figs. 56 to 59 and 61 tend to validate this statement, while the other cases, especially those in Fig.60, do not.
2. The writers believe that because of differences in sensitivitiy and the $\sigma_p'$ mechanism among the different soil deposits, no unique relationship between $B_q$ and changes in OCR should be expected.
3. In cohesive deposits with a highly developed macro-fabric like the Porto Tolle silty clay, the use of a $B_q$ vs depth trend as an indicator of OCR may be problematic.
4. The $B_q$ is simultaneously an index of the stress history and of soil type (see section 3.2.2), and therefore it supposedly reflects not only OCR changes but also local soil heterogeneity.
5. In conclusion, the writers would hope that $B_q$ reflects OCR changes within a given uniform cohesive deposit. However an increase of the reliability of the $B_q$ = f (OCR) correlations and its further validation will require:
   - a much deeper insight into the incremental total stress field around the CPTU tip; see Baligh (1984);
   - standardization of the device and the test procedure;
   - accumulation of a large number of reliable experimental CPTU data obtained in geotechnically well-documented deposits of different types.

### 3.2.10. Summary and Conclusions

#### Soil Profiling and Identification

1. The in situ techniques have a great potential for soil profiling and identification.
2. Among these techniques the piezocone (CPTU) measuring continuously the $q_c$, $u_{max}$, and $f_s$ seems to be particularly promising and powerful. However, one has to recognize that the use of the piezocone in practice is not so straightforward as it appears, at least in principle.
   Particular attention and care should paid in order to assure that sufficiently rigid and properly de-aired measuring system is used.
   At least two research needs for CPTU should be recognized:
   - greater insight into the theories controlling the distribution of $u_{max}$ along the penetrating tip and above it;
   - the relevance of the characteristics of the filters and saturation fluids to the measured pore pressure response, and the influence of the location of the porous stone.

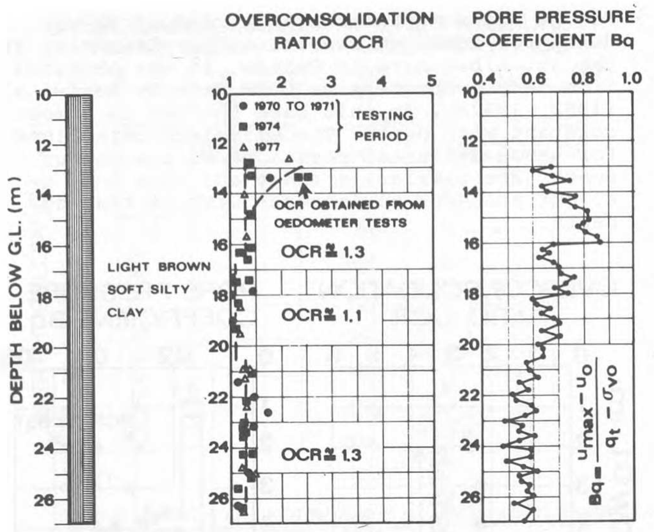



Fig.60: Pore Pressure Coefficient $B_q$ vs. OCR in Soft Silty Clay at the Porto Tolle Site.

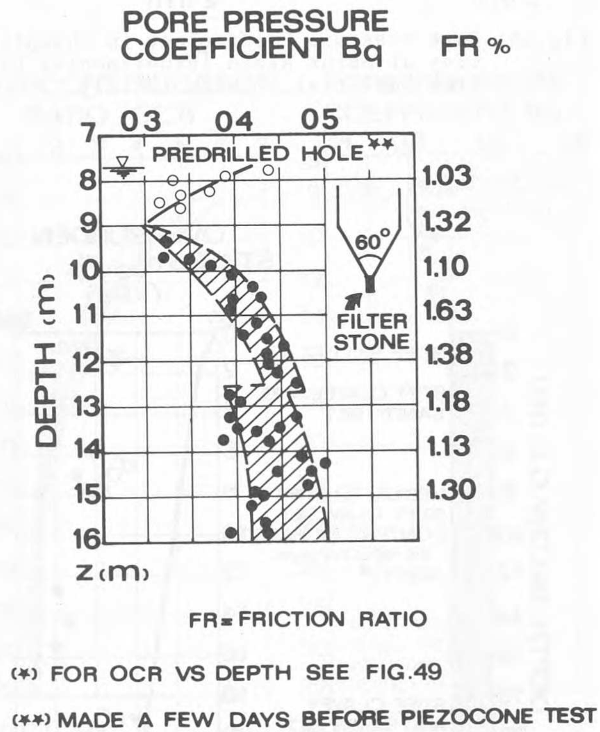



Fig.61: Pore Pressure Coefficient $B_q$ in Heavily OC Cemented Taranto Clay*.

3. The present positive experience with well deaired CPTU is limited mostly to cohesive deposits. Further experience will be required in order to recognize and understand the advantages and limitations in using of piezocones in sands.

4. Various identification charts based on the CPT, CPTU and DMT are available in the geotechnical literature. It must be realized that they are able to yield only approximate information on soil type, reflecting mainly the differences between granular and fine grained soils and hence the difference in soil permeability.

In Situ Horizontal Stress and Stress History

1. The reliable assessment of the existing in situ horizontal stress represents a challenging and not yet completely solved problem of ESE.

2. Among the presently available in situ techniques, the SBP seems to be most promising, at least in clays.

3. Experience in the use of the SBP to assess $\sigma_{ho}$ in sands is still very limited and mostly negative; this is mainly due to disturbance caused by the insertion of the probe which leads to the measured $\sigma_{ho}$ being generally too low but close to the equilibrium $u_o$.

4. In order to increase the reliability of the values of $\sigma_{ho}$ measured by the SBP, further improvement of the probe mechanics and electronics and of the insertion procedures is warranted. In addition, the SBP probe must be inserted with a perfectly cylindrical shape.

5. The experience with flat spade-like cells for measuring $\sigma_{ho}$ in soft and more recently even in stiff clays looks promising. However in stiff clays, there is the problem of correcting the "raw" readings for bedding errors.

6. The DMT yields reasonable values of $K_o$, hence $\sigma_{ho}$, in soft and medium noncemented clays having $I_L < 1$. Limited experience with a newly developed procedure for sands shows quite promising results, at least in distinguishing between cohesionless deposits with "normal" lateral stress from those having "high" lateral stress. However, due to its empirical nature, it requires validation for a wide range of typical sands and silty sands. The procedure is especially attractive if one considers the cost-effectiveness of the DMT.

7. The pore pressure coefficients obtained from CPTU do reflect changes in the OCR profile in clays. This is especially true in uniform deposits. In deposits with a highly developed macro fabric, the local variability in the soil permeability makes this approach less sensitive.

## 3.3. DEFORMATION PARAMETERS

### 3.3.1. General Aspects

The determination of soil deformation parameters from in situ tests is of great practical interest because of:

- the well known limitations of laboratory methods when dealing with heavily OC clays and sands; it is extremely difficult to obtain high quality undisturbed samples of these materials;
- the well known sensitivity of the deformation modulus to even small disturbance, especially at low strains [Simpson et al. (1979), Jardine et al. (1984)].

These factors make the laboratory tests either inapplicable (cohesionless soils) or of doubtful reliability (cohesive deposits) if not performed on very high quality undisturbed block samples. However, when performing in situ tests in which the main goal is to obtain the soil deformability parameters, the following important points should be kept in mind; these points affect both the interpretation of the specific in situ test and use of these parameters in engineering analyses:

1. Each of the soil deformation parameters depends on the octahedral effective stress $\sigma'_{oct}$ and the stress history of the soil surrounding the in situ device at the beginning and during the test, i.e.:

$$E \text{ or } G \text{ or } M = f(\sigma'_{oct}, OCR)$$

where:
E = Young's modulus
G = shear modulus
M = constrained modulus
Stress changes caused by the insertion of the device must also be included.

2. Because of the nonlinear and elasto-plastic nature of soil behaviour, both E and G are strongly dependent on the in situ shear stress level at which they are assessed.

3. The drainage conditions of any in situ test in relation to the permeability and existing free drainage boundaries of the soil should be realistically assessed in order to correctly interpret the experimental data with respect to the derived deformation modulus.

4. The majority of soil deposits are not isotropic; therefore the deformation moduli obtained from in situ tests reflect both the degree of anisotropy of the soil and the stress path followed.

As far as this last problem is concerned, it has been shown experimentally that because of the depositional and consolidation stress history of many natural soil deposits, their pre-failure stress-strain response may be adequately described by the cross-anisotropic elasticity model [Lekhnitskii (1977), Gibson (1974), Graham and Houlsby (1983)].
The practical relevance of this physical situation is that the deformation modulus obtained from a test which stresses the soil in a vertical direction, for example PLT, leads to stiffness values of $E_v$, $G_{vh}$ which may be appreciably different from those such as $E_h$, $G_{hh}$, which are deduced from SBP which stresses the soil in a horizontal direction.
However, relatively little experimental data are available as to the influence of anisotropy on the different deformation moduli of natural soil deposits. This is especially true for parameters describing the drained soil deformability.

### 3.3.2. Deformation Moduli from Self-Boring Pressuremeter (SBP)

The present section briefly analyses the problem of determination of soil deformation moduli from SBP tests. The objective is to evaluate the potential of this device to give parameters which are useful in the calculation of vertical settlements and lateral displacements of soils. If a long pressuremeter probe is expanding in a elastic-perfectly plastic soil, the surrounding soil is only subjected to shear. Thus it must be realized that the test measures only the shear modulus of soil G.

The relevant formulae for the computation of G are [Gibson and Anderson (1961), Hughes (1973), Wroth (1975), Windle (1976), Mair and Wood (1985)]:

$$G = \frac{dp}{d\varepsilon_v} \quad \text{or} \quad G = \frac{1}{2}\frac{dp}{d\varepsilon_\theta}$$

where:

$dp$ = $p - \sigma_{ho}$ = change of radial cavity stress from the reference stress state; the latter is equal to the total horizontal in situ stress prior to insertion of the probe

$d\varepsilon_\theta$ = corresponding change of the circumferential strain of the cavity $\varepsilon_\theta = (r_c - r_o) / r_c$

$d\varepsilon_v$ = corresponding change of the volumetric strain of the cavity $\Delta V / V_c = (V_c - V_o) / V_c$

$u_c$ = lateral displacement of the cavity wall

$V_c$ = current volume of the pressuremeter probe

$V_o$ = initial volume of the pressuremeter probe at the reference stress state

$r_c$ = current radius of the expanding cavity

$r_o$ = initial radius of the probe at the reference stress state

Three different shear moduli may be computed from the SBP test results (Fig.62):

1. Pressuremeter shear modulus from the expansion curve, $G_i$ (initial tangent) or $G_s$ (secant).

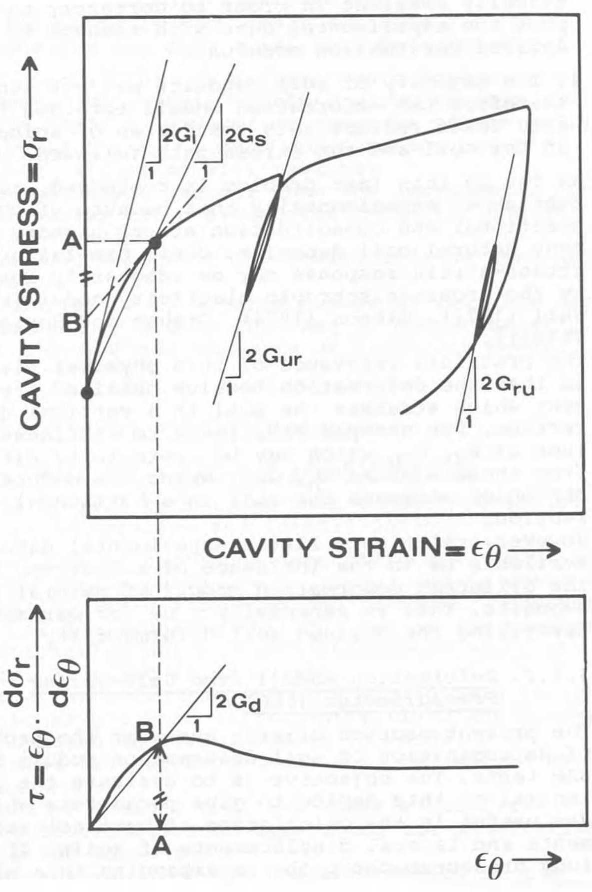



Fig.62: Illustration of the Determination of the Three Shear Moduli from the SBP Test.

2. Pressuremeter shear modulus from the unload-reload loop, $G_{ur}$ performed during the loading stage, or $G_{ru}$ from reload-unload loop carried out in the unloading stage.

3. Shear modulus computed from the stress-strain curve derived from the expansion curve [Baguelin et al. (1972), Wroth and Hughes (1972, 1973), Palmer (1972), Ladanyi (1972), Denby (1978), Ladd et al. (1979), Ghionna et al. (1981)], called here the derived modulus, $G_d$. At present this last approach is applicable only to a quick expansion test performed in saturated cohesive deposits, so it occurs in undrained conditions.

Among these three possibilities, the unload-reload loop modulus $G_{ur}$ seems to be considered as the most reliable [Windle (1976), Simpson et al. (1979), Parry (1979), Wroth (1982), Hughes (1982), Fahey and Randolph (1984), Mair (1984), Mair and Wood (1985)]. The reasons for this opinion are summarized as follows:
The experience gained at MIT and TUT [Ladd et al. (1979), Ghionna et al. (1981, 1982)] in use of the SBP showed that all the methods used to obtain the complete stress-strain curve from quick (undrained) expansion tests in cohesive soils lead to "derived curves" which are not necessarily reliable. The shape of the "derived curves" and hence the obtained parameters are influenced by factors like:

- Even small disturbance of the surrounding soil [Baguelin et al. (1978)].
- Error in the estimated reference stress state, $\sigma_{ho}$ [Schmertmann (1975), Germaine (1980), Ghionna et al. (1981, 1982, 1983)].
- Numerical instability in the derivation process used to obtain the $\tau$-$\gamma$ curve from the expansion curve [Baguelin et al. (1972), Ladd et al. (1979), Ghionna et al. (1981)]. Because of these problems, it is necessary to "smooth" the expansion curve by a more or less arbitrarily selected fitting equation [Baguelin et al. (1972), Jamiolkowski and Lancellotta (1977), Denby (1978), Ladd et al. (1979)].

In addition to the above mentioned factors, phenomena like partial drainage and large variation of the strain rate with radial distance from the cavity may cast doubts about the reliability of "derived" stress-strain curves for clays.
With respect to G this lack of reliability is reflected in more or less pronounced differences between "derived" $G_{d50}$ and $G_s$ evaluated from the expansion curve at the cavity strain $\varepsilon_{\theta 50}$.
Here:

$G_{d50}$ = shear modulus at a stress level equal to half the maximum shear stress at failure computed from the derived stress-strain curve

$G_s$ = secant pressuremeter shear modulus computed at $\varepsilon_{\theta 50}$

$\varepsilon_{\theta 50}$ = circumferential strain on the derived stress-strain curve corresponding to the stress level equal to half of the maximum shear stress

When performing the unload-reload loop to obtain $G_{ur}$, care must be exercised not to exceed the "elastic" limit of the material during unloading. For elastic-perfectly plastic materials, the allowable magnitude of unload (for both drained and undrained expansion tests) which theoretically will prevent the soil from failing at the cavity wall (when $\sigma_\theta = \sigma_1$ and $\sigma_r = \sigma_3$) has been given by Wroth (1982):

- in drained tests: $\Delta p_{max} \leq \frac{2 \sin \phi'}{1 - \sin \phi'} (p - u_o)_{max}$

- in undrained tests: $\Delta p_{max} \leq 2\ c_u$

where:

$\phi'$ = drained peak angle of shearing resistance
$c_u$ = peak undrained shear strength
$\sigma_\theta$ = circumferential stress
$\sigma_r$ = radial stress = p
$(p - u_o)_{max}$ = the effective radial cavity stress at the start of the unload cycle

Since the real soil very often behaves differently from the postulated idealized elastic-perfectly plastic material, Mair and Wood (1975) recommended the use in practice of lower $\Delta p$ values rather than those resulting from the above theoretical criteria. This is particularly appropriate in clays if a significant hysteresis in the unload-reload cycles is to be avoided.

Figure 63 and Table X show the $G_{ur}$ and $G_{ru}$ as obtained from SBP tests performed in the slightly silty sand from the Po River Valley. An analysis of these data shows that on the average, there is a tendency of $G_{ur}$ to increase moderately as $p_{max}$ increases. This may reflect either an increase of $\sigma'_{oct}$ in the surrounding soil or a stiffening of the sand due to shear prestraining experienced during previous unload-reload cycles. Comparison of $G_{ur}$ with $G_{ru}$ values indicates that the latter are close to the average value of the shear modulus obtained from the unload-reload loops.

Figs. 64 and 65 shows the value of $G_{ur}$ and $G_s$ measured in two soft Italian clays [Ghionna et al. (1982, 1983)] compared with the $G_{50}$ obtained from laboratory tests on good quality undisturbed samples. Fig.66 shows the same type of comparison for the heavily OC micro-fissured cemented Taranto clay [Ghionna et al. (1983)].

TABLE X

SBP "Elastic" Shear Modulus as Obtained in Medium Dense to Dense Slighthly Silty Sand of the Po River Valley (Hole No.3)

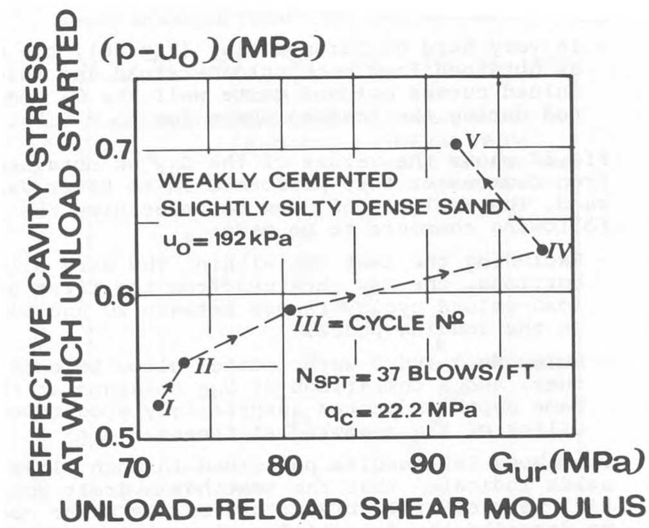
| DEPTH | $G_{ur}$ (MPa) | | | | | $G_{ru}$ (MPa) |
|---|---|---|---|---|---|---|
| | CYCLE | | | | | |
| (m) | I | II | III | IV | V | |
| 3.2 | 9.7 | - | - | - | - | 7.5 (a) |
| 4.7 | 18.2 | - | - | - | - | 18.3 |
| 6.2 | 25.1 | 26.6 | - | - | - | 26.6 |
| 7.7 | 24.6 | - | - | - | - | 21.6 (a) |
| 9.2 | 31.1 | 33.7 | 38.8 | 40.0 | - | - |
| 10.7 | 40.7 | 40.7 | - | - | - | - |
| 12.2 | 50.0 | 55.5 | - | - | - | 47.0 |
| 15.2 | 57.2 | 51.1 | - | - | - | - |
| 16.7 | 53.2 | 58.9 | 59.0 | - | - | 57.9 |
| 18.2 | 47.7 | 49.0 | - | - | - | 52.3 |
| 19.7 | 45.7 | 57.9 | - | - | - | 49.1 |
| 21.2 | 71.6 | 73.0 | 80.4 | 98.1 | 92.0 | - (b) |
| 23.2 | 23.2 | 24.9 | 25.8 | - | - | - (c) |

(a) More Silty
(b) Slight Cementation
(c) Sandy Silt

$G_{ur}$ = unload-reload shear modulus
$G_{ru}$ = reload-unload modulus
The $(p - u_o)_{max}$ increases with increasing number of the cycle

These experimental results allow the following comments to be made:

- Any type of G computed from the SBP is generally much higher than those obtained from laboratory tests. The difference is so large that it cannot be explained by factors like clay anisotropy or the differences in strain rates or stress paths between SBP and laboratory tests.




| CYCLE Nº | $p_{max}$ kPa | $\Delta p$ kPa | $\Delta \epsilon_v$ % |
|---|---|---|---|
| I | 52 6 | 10 2 | 0.05 |
| II | 55 3 | 13 0 | 0.08 |
| III | 58 9 | 16 5 | 0.10 |
| IV | 65 9 | 21 4 | 0.15 |
| V | 70 7 | 28 1 | 0.18 |

$p_{max}$ = TOTAL CAVITY STRESS WHEN UNLOAD STARTS
$\Delta p$ = LOAD DECREMENT
$\Delta \epsilon_v$ = VOLUMETRIC STRAIN DURING LOOP
$\Delta p_{max} = 3.2\ (p_{max} - u_o)$

Fig.63: Example of $G_{ur}$ Variation in Sand of the Po River Valley with Cavity Stress and Number of Cycles.

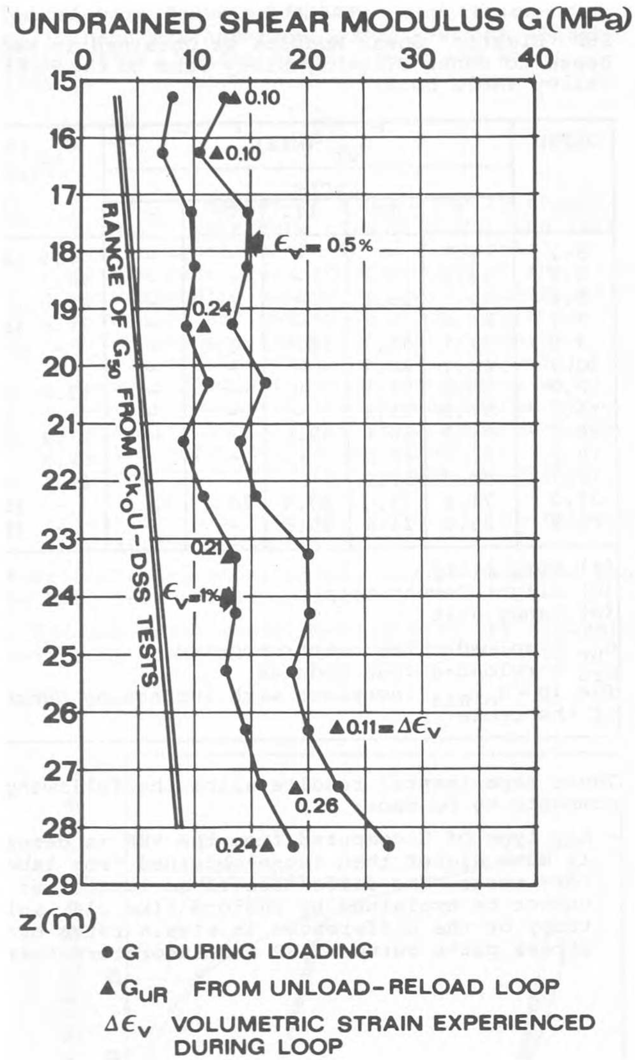



Fig.64: Shear Modulus from SBP at the Porto Tolle Site.

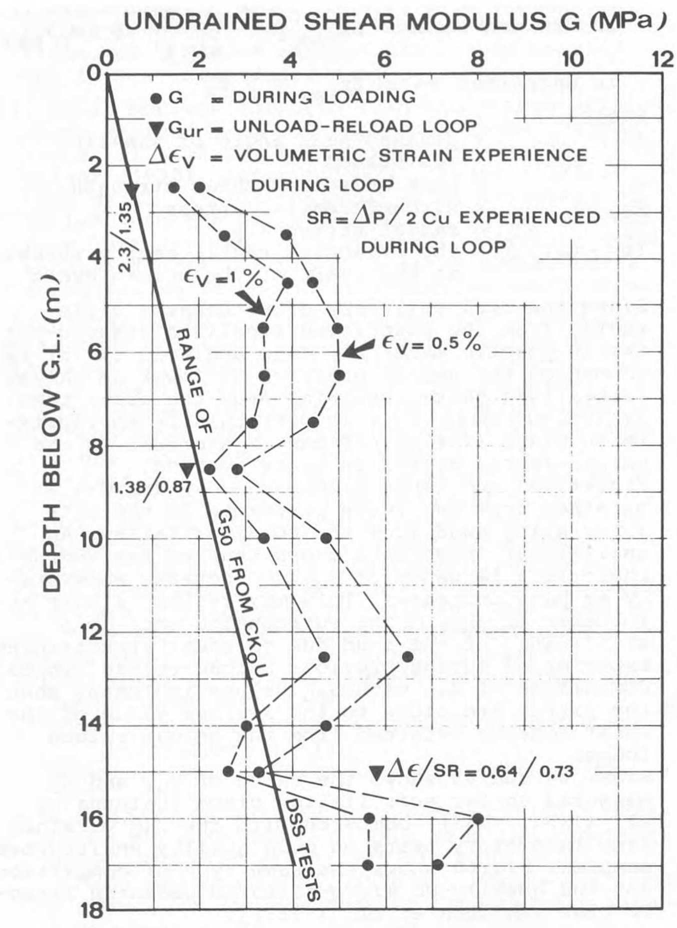



Fig.65: Shear Modulus from SBP at Panigaglia Site.

Considering the well known sensitivity of G to even small disturbance [Simpson et al. (1979), Costa Filho (1979), Jardin et al. (1984)] and the observed differences between G (SBP) and G (Lab), there is some doubt about the reliability of laboratory-determined moduli [see also Windle and Wroth (1977)].

- When a comparison is made between $G_s$ and $G_{ur}$ in the three clays tested, it appears that there is generally good agreement between these two moduli, if the $G_s$ is computed for a $\varepsilon_v$ ranging between 0.5 and 1%.
- For the two soft clays in Figs.64 and 65, the influence of the volumetric strain $\Delta\varepsilon_v$ experienced by the cavity during the unload-reload loop is evident. The $G_{ur}$ tends to decrease while $\Delta\varepsilon_v$ increases.
- In very hard OC Taranto clay (Fig.66), the $G_{ur}$ as obtained from both unload-reload and reload-unload curves matched quite well the $G_s$ computed during the loading phase for $\varepsilon_v$ = 0.5%.

Fig.67 shows the values of the $G_{ur}$ as obtained from Camkometer test performed in Po River Valley sand. The analysis of these tests enables the following comments to be made:

- Excluding the test run within the more silty horizons, the $G_{ur}$ obtained from the first unload-reload cycle, ranges between 20 and 50 MPa in the loading phase.
- Holes No.1 and 2 were located close to each other, and a comparison of $G_{ur}$ obtained at the same depth indicated surprisingly good repeatability of the measured stiffness.

The above SBP results performed in both clays and sands indicate that the test has a great potential to yield reliable values of the shear moduli to describe the "elastic" soil response under induced shear stresses. When using SBP shear moduli to estimate "elastic" ground deformations, the designer must pay due attention to the following points:

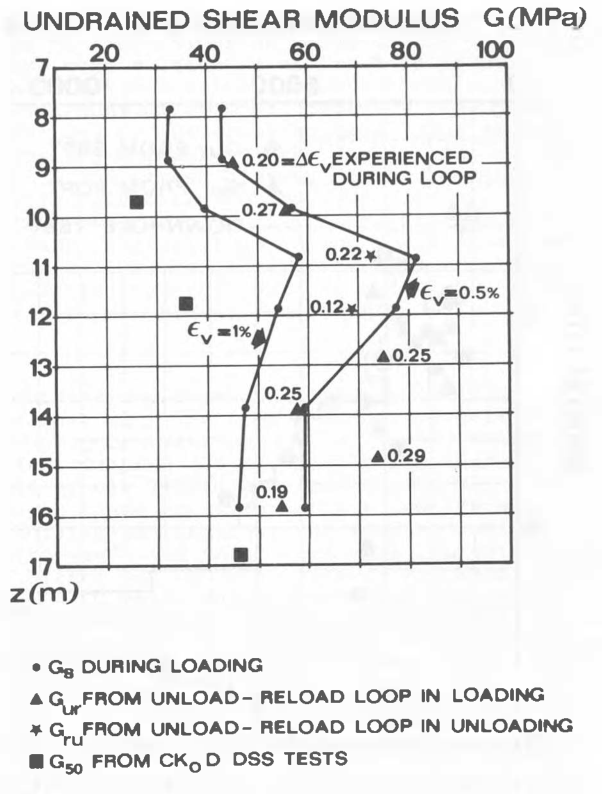



Fig.66: Shear Modulus from SBP at the Taranto Site.

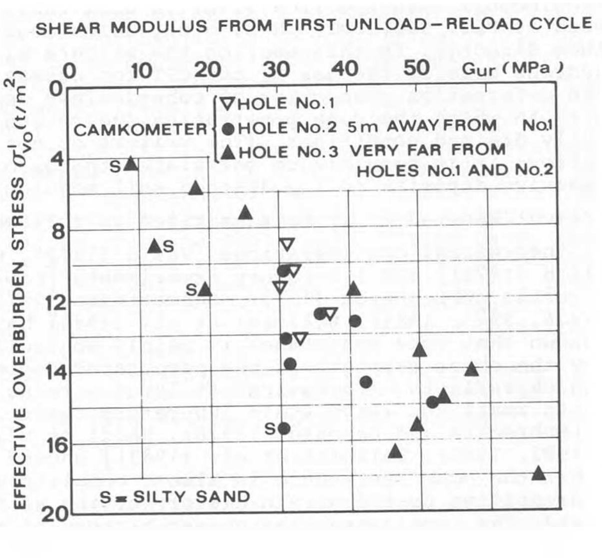



Fig.67: Shear Modulus of Medium Dense to Dense Sand of the Po River Valley as Obtained from SBP Tests.

1. Because of the assumptions (infinitely long cylindrical cavity expanded in elastic-perfectly plastic soil; consequently $\Delta\sigma_{oct} = 0$, $\Delta u = 0$) generally adopted for the interpretation of the SBP, the measured G should be independent of the drainage conditions [Mair and Wood (1985)]. If this is true for isotropic clays, it is possible, at least in principle, to convert G to $E_u$ and E' using the following formulae:

$$E_u = 2\,G\,(1 + \nu_u)$$

$$E' = 2\,G\,(1 + \nu')$$

where:
$E_u$ = undrained Young's modulus
E' = drained Young's modulus
$\nu'$ = drained Poisson's ratio (0.10 to 0.30) [see Mair and Wood (1985)]
$\nu_u$ = undrained Poisson's ratio (0.5)

However this approach cannot be generally applicable since the behaviour of soil often deviates significantly from the assumed idealization. The most relevant deviations are:
- In many cohesive deposits and especially in soft clays, shear stresses generate excess pore pressure. Ladd and Edgers (1972) reported for lightly OC clays (OCR $\leq$ 1.5 to 3) a positive $\Delta u$ resulting in $CK_oU$-DSS at low shear strain levels to which corresponded relatively little $\Delta\sigma_{oct}$. In highly OC clays, negative $\Delta u$ have been observed in similar circumstances.
- Since natural soils are seldom isotropic, the obtained $E_u$ and E' may have limited significance regarding in situ stress-strain behaviour, with the exception of problems involving similar stress paths.

From the above it is possible to tentatively postulate that in order to obtain reliable values of $E_u$ or E' from the pressuremeter shear modulus, the following conditions must be met:
- Overconsolidated preferably lean clay deposit.
- $G_{ur}$ determined from a unload-reload cycle run at an early stage of the expansion test.

2. Because SBP shears the soil in a horizontal direction, it yields a shear Modulus $G_{hh}$ [Livneh et al. (1971), Hartman (1974)]. In that circumstance if one is interested in the $G_{vh}$ value corresponding to deformation on vertical planes, it is necessary to use relationships between the various elastic constants given, e.g., by Warren (1982) and Graham and Houlsby (1983).

3. All of the soil deformation parameters are very sensitive to the actual strain level of the soil macro-element being considered. Therefore it is essential that in any deformation calculations, the adopted moduli be determined in the appropriate range of the strain or stress level anticipated for the specific design. Unfortunately, except the case of clays when obtaining the derived $G_d$ from $\tau$-$\gamma$ curves [Baguelin et al.(1972) Palmer (1972) and Ladanyi (1972)], there are no rigorous means to relate the measured pressuremeter moduli to either strain or stress level of a representative soil element.

4. As already mentioned the $G_{ur}$ in sands reflects soil behaviour in the "elastic" region below the yield surface. This enables the direct use of $G_{ur}$ in deformation computations under the following circumstances:
- either the calculation is performed using an incremental elasto-plastic soil model in which elastic and plastic deformations are evaluated separately;

- or the calculations are performed using a simplied soil model when the stress levels fall below the yield locus.

5. An assessment of the relevance cal computations of the $G_{ur}$ obtained in cohesive deposits appears more difficult. In this case due to the possible presence of excess pore pressure, its corresponding position in effective stress space is not exactly known.
Based on the experience gained in UK [Burland et al. (1977), Simpson et al. (1979), Tedd et al. (1984), Mair and Wood (1985)] it may be argued that the SBP $G_{ur}$ is appropriate for direct use in design calculations involving small strain problems in stiff and hard clays. However in this case, due consideration should be given to the strain level at which the Gur has been computed. It should be compared at the strain level anticipated in the engineering application.
At least in principle, pressuremeter tests run with the PM and PIP may lead to a reliable evaluation of an "elastic" shear moduli from properly programmed unload-reload loops. Since insertion of these types of pressuremeters involves significant disturbance of the surrounding soils, it is advisable to perform the unload-reload cycle at a late stage of the loading phase, so that the installation effects are hopefully minimized. Data concerning the determination of the $G_{ur}$ from the PM and PIP tests may be found in Marsland and Randolph (1977), Powell et al. (1983), Frydman (1979), Baguelin et al. (1978), Mair and Wood (1985). Hughes and Robertson (1984) reported $G_{ur}$ values obtained from tests in a well-documented sand deposit run with a newly developed Full Displacement Pressuremeter (FDP). The $G_{ur}$ values from the FDP are in quite good agreement with those obtained from the SBP. They are also comparable to the G values determined from shear wave velocity measurements using the down-hole technique (see Fig.68). These authors also give a qualitative theoretical justification why a supposedly good agreement should exist between the $G_{ur}$ values obtained from the two types of pressuremeter tests. However, because the unload-reload cycle is performed at a late stage of the expansion tests using PMT, PIP, and FDP, they may lead to an alteration of the initial "structure" of the soil due to large prestraining. Thus the reliability of $G_{ur}$ values obtained from these tests requires further experimental validation.

### 3.3.3. Deformation Moduli from Cone Penetration Test (CPT)

For forty years engineers have tried to correlate the cone resistance with different soil deformation moduli in order to be able to compute the settlements of foundations [De Beer and Martens (1957), Trofimenkov (1974), Schmertmann (1972), Sanglerat (1972), Dahlberg (1975)]. For further references, see Mitchell and Gardner (1975), Schmertmann (1978), Campanella and Robertson (1983), Robertson and Campanella (1984) and Meigh (1985).
All these attempts may be grouped into two basic categories:

1. Methods trying to correlate the observed settlements of existing structures with $q_c$ measured before construction [Schmertmann (1972), Schmertmann et al. (1978)].

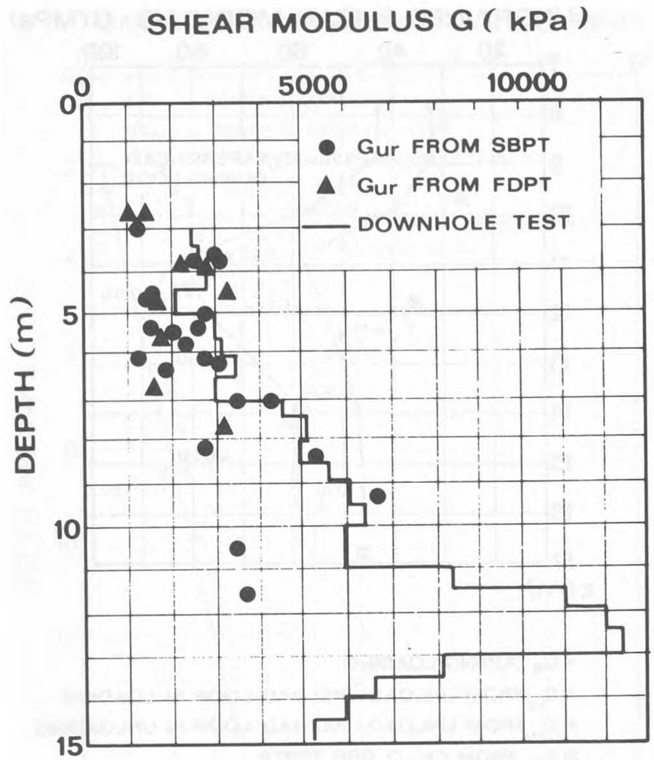



Fig.68: Comparison of Shear Moduli as Obtained in Silty Sand at McDonald Farm Site from Pressuremeter and Down-Hole Tests [Hughes and Robertson (1984)].

2. Correlations based on a comparison between field measured $q_c$ and laboratory assessed deformation moduli.

Surprisingly little effort has been spent in attempts to correlate the $q_c$ against the deformation moduli obtained from other in situ tests such as PLT, SBP, PMT, CHVs, etc., which measure them directly. In this section the writers will address briefly the use of the CPT for assessing the deformation properties of cohesionless deposits in which the cone penetration occurs under fully drained conditions. (The writers do not believe it is possible to correlate the $q_c$ of cohesive deposits to any drained soil modulus).

Present knowledge may be summarized as follows:

1. Theoretical considerations [Vesic (1972), Baligh (1975)] and laboratory experiments [Lambrechts and Leonards (1978), Schmertmann (1972, 1976, 1977, 1984), Bellotti et al. (1983) have shown that cone resistance is mainly controlled by the shear strength of the penetrated deposit, which reflects its behaviour at large strains. Both small and large scale laboratory tests [Lambrechts and Leonards (1978), Baldi et al. (1982, 1985), Bellotti et al. (1983)] showed that the cone resistance is almost completely insensitive to the strain history of the tested sand. The cone senses the stress history of the sand to some extent only as a function of changes in $\sigma_h'$ resulting from mechanical overconsolidation [Schmertmann (1972, 1977, 1978), Baldi et al. (1985)].

2. The first approach linking the observed settlements of foundations on sands to the $q_c$ measured before construction is at least in principle of great practical value. However these types of correlations are strongly dependent on specific conditions of the examined case, such as load intensity, geometry of the foundation, characteristics and stress history of the subsurface profile, and construction schedule. All these factors make it difficult to generalize this approach, unless it is on the basis of a statistical analysis of a large number of well documented case records [See Burland et al. (1977) as far as the SPT is concerned].

3. Correlations between $q_c$ and deformation moduli have been obtained from a number of calibration chamber (CC) tests by Holden (1971), Veismanis (1974), Chapman (1979), Parkin and Lunne (1982), Baldi et al. (1982) and Bellotti et al. (1983). However these experiments have been done on artificially-sedimented clean fine to medium quartz sand, and this poses some limitations on the direct use of these data in practice. Moreover the CC tests showed that even for the same tested sand no unique correlation between $q_c$ and E' or M exists. Carefully prepared and conducted laboratory experiments showed that both $E'/q_c$ and $M/q_c$ depend the factors like:

- Stress and strain history of the CC specimens.
- Their relative density.
- Mineralogical composition of the test sand which is related to the crushability of the grains.

From the above, it appears that empirical correlations between $q_c$ and E' or M cannot be considered either highly reliable or of general validity.
Despite these limitations, such correlations are used and will probably continue to be used by designers, mainly because the CPT is widely available, and there is a lack of technically and economically valid alternatives (an exception may be the DMT).
However, even given this situation, the writers think it worthwhile to report ratios of $E'_{25}/q_c$ based on $q_c$ measured in CC tests performed on pluvially-deposited specimens of Ticino and Hokksund sands [Parkin and Lunne (1982), Baldi et al. (1982), Bellotti et al. (1983), Lunne and Christofferssen (1984)]. The values of $E'_{25}$ have been determined from the $CK_0D$ TC tests on pluvially-deposited specimens of the same sands. From the summary of the $E'_{25}/q_c$ ratios given in Table XI, it may be observed that for NC Ticino Sand, they agree quite well with those suggested by Schmertmann (1972) and Schmertmann et al. (1978). The same ratios for NC Hokksund Sand are slightly lower mainly due to the lower Young Moduli from the $CK_0D$ TC tests.
The writers would emphasize that much higher $E'_{25}/q_c$ ratios have been obtained for both sands when overconsolidated. This fact suggests the following:

1. The cone resistance is much less sensitive to the stress history of the sand than the $E'_{25}$.
2. The use of $E'_{25}/q_c$ ratios appropriate for NC deposits to compute the settlement of foundations resting on sands which have been subjected to any kind of prestressing or prestraining will lead to gross overestimates.

TABLE XI

$E'_{25}$ vs $q_c$ from CC Tests on Ticino and Hokksund Sands at Different Relative Densities

| Sand | OCR | $D_R$=30% | $D_R$=50% | $D_R$=70% |
|---|---|---|---|---|
| Ticino | 1 | 2.6 | 2.4 | 2.2 |
| Hokksund | 1 | 2.3 | 2.0 | 1.8 |
| Ticino | $\geq$ 2 | 19.2 | 14.0 | 10.1 |
| Hokksund | $\geq$ 2 | 18.9 | - | 5.7 |

With reference to the SBP tests, mentioned in Section 3.3.2., which were run by TUT [Ghionna (1984)] in medium dense to dense (55% $\leq D_R \leq$ 70%) slightly silty sand of the Po River Valley, the writers compared the $G_{ur}$ obtained from the first unload-reload cycle to $q_c$ measured with the electrical cone at the distance of ≅ 5 m from the pressuremeter holes. The comparison was restricted to tests performed at elevations at which the grain size distribution of the soil was also determined in laboratory, i.e., only to tests yielding supposedly fully drained $q_c$ and $G_{ur}$ values. The average value of the $G_{ur}/q_c$ obtained for 14 SBP performed in sands with the fraction passing sieve N°200 ASTM ranging between 4% and 10% is:

$$G_{ur} / q_c = 3.80 \pm 0.85$$

If one adopts a Poisson's ratio $\nu' = 0.3$, this leads to a ratio between unload-reload Young modulus of:

$$E_{ur} / q_c = 13.6 \pm 2.2$$

This value falls within the range of $E'_{25}/q_c$ ratios obtained for the OC Ticino Sand and Hokksund sands (Table XI).
Since the Po River Sand is presumably NC or only lightly OC, the resulting $E_{ur}/q_c$ ratio probably reflects the influence of the shear prestraining experienced by the sand during the pressuremeter unload-reload cycle. This leads to a soil response similar to that observed for OC sands.
The other approach to estimate sand deformability on the basis of the CPT data consists of an evaluation of the $D_R$ as function of $q_c$ and $\sigma'_{vo}$ [Schmertmann (1976), Lunne and Christoffersen (1984), Baldi et al. (1983, 1983a)].
Once $D_R$ is assessed E' or G can be estimated as a first approximation, at least for NC sands, as a function of $D_R$.
In order to help the designers in that respect, Fig.69 reports the correlation between $D_R$ and $q_c$ through $\sigma'_{vo}$ worked-out by Lancellotta (1983) from a regression analysis of well-documented and comprehensive CC tests data for NC sands available in the literature [Holden (1971), Harman (1976), Schmertmann (1978), Parkin et al. (1980), Baldi et al. (1981, 1982), Lunne and Christoffersen (1984)]. This correlation has been obtained using the following assumptions:

- A linear relationship is postulated between $D_R$ and $\log_{10} [q_c / (\sigma'_{vo})^{\alpha}]$ from data from pile load tests [Vesic (1977)].
- Assuming as a first approximation that $\alpha = 0.5$ [Baldi et al. (1985)].
- Postulating that in the CC tests the uncertainties regarding $q_c$ and $\sigma'_{vo}$ are negligible as compared to those affecting $D_R$.

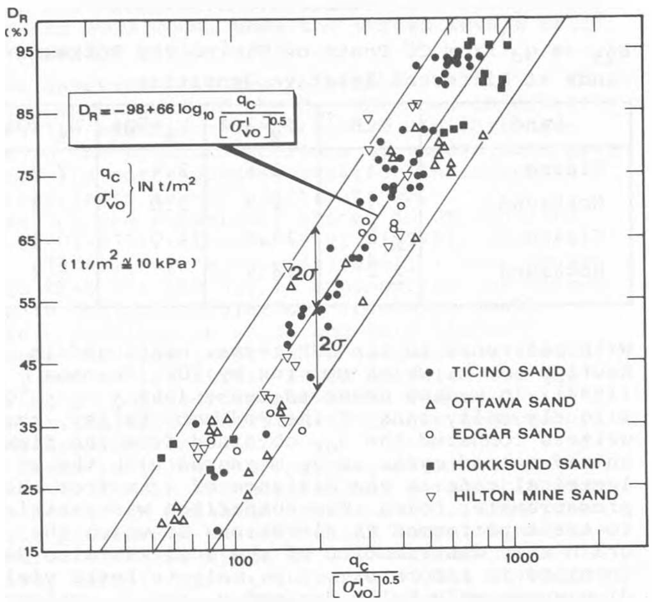



Fig.69: Correlation Between $D_R$ and $q_c$ Through $\sigma'_{vo}$ [Lancellotta (1983)].

The regression analysis of CC data is based on the following relationship:

$$D_R = A + B \log_{10} \left[\frac{q_c}{\sqrt{\sigma'_{vo}}}\right]$$

with the $D_R$ values "weighted" by $1/Var[D_R]$, in which the variance $Var[D_R]$ is the one obtained by Rossi (1983) for Ticino Sand.
The regression (A, B) and correlation (R) coefficients are shown in Table XII and the results of this analysis are summarized in Fig.69.

TABLE XII

Regression Coefficients for $D_R$ vs. $q_c$ correlation [Lancellotta (1983)]

| Sand | $\hat{A}$ | $\hat{B}$ | $\hat{\sigma}$ | R | Number of data points |
|---|---|---|---|---|---|
| Ticino Sand | - 108 | 71 | 5.2 | 0.96 | 46 |
| All 5 Sands | - 98 | 66 | 6.6 | 0.96 | 144 |

The correlation is applicable to NC uncemented, unaged sands in which quartz minerals are predominant and for which it may be assumed that the measured $q_c$ corresponds to the fully drained cone resistance. The correlation is not directly applicable to OC sand deposits for which no unique relationship exists between $D_R$ and $q_c$ through $\sigma'_{vo}$ [Schmertmann (1976), Baldi et al. (1983, 1983-a, 1985)]. The obtained $D_R$ values may be corrected for the chamber size effect [Parkin and Lunne (1982)] by dividing the field measured $q_c$ value by a factor $K_q$ from the following formula:

$$K_q = 1 + \frac{0.2\,(D_R - 30)}{60}$$

This formula is the same as the one used by Lunne and Christoffersen (1984) but considers the chamber size effect as assessed experimentally from the CC test data obtained for the Ticino and Hokksund Sands. The chamber size effect will otherwise lead to an overestimate of $D_R$; the amount increases as $D_R$ increases.
[More details concerning the use of CPT for evaluation of deformability of the cohesionless deposits may be found in Senneset et al. (1982), Campanella and Robertson (1983), and Robertson and Campanella (1984)].

### 3.3.4. Deformation Moduli from Marchetti Flat Dilatometer (DMT)

The DMT yields the dilatometer modulus $E_D$ which is given by the following formula [Marchetti (1980)]:

$$E_D = \frac{E}{1 - \nu^2} = 38.2\,(p_1 - p_o)$$

where:

$E$ = Young modulus of elasticity
$\nu$ = Poisson's ratio

The $E_D$ was developed by Marchetti (1980) from the Theory of Elasticity solution for a uniformly loaded flexible circular area on the surface of an elastic half-space with zero normal displacement outside its perimeter. The first attempt to correlate $E_D$ to the constrained tangent modulus M was through an empirically established proportionality coefficient $R_M = M / E_D$. Based on experimental evidence from different well documented Italian soil deposits, Marchetti (1980) proposed the following rules to evaluate $R_M$ as a function of $I_d$ and $K_D$ (see section 3.2.3):

$$I_D \leq 0.6: \quad R_M = 0.14 + 2.36 \log_{10} K_D$$
$$0.6 < I_D \leq 3.0: \quad R_M = R_{MO} + (2.5 - R_{MO}) \log_{10} K_D$$
$$I_D \geq 3.0: \quad R_M = 0.5 + 2 \log_{10} K_D$$

where: $R_{MO} = 0.14 + 0.36 \cdot \frac{I_D - 0.6}{2.4}$

If these rules yield $R_M < 0.85$, then use $R_M = 0.85$.

According to Marchetti (1980), the product of RM . $E_D$ corresponds to the tangent contrained modulus at the effective overburden stress $\sigma'_{vo}$ existing prior to the insertion of the dilatometer. A number of researchers have performed DMT in a variety of soil deposits [Lacasse and Lunne (1982b), Hayes (1983), Campanella and Robertson (1983), Aas et al. (1984)]. They report comparisons between $E_D$ and M showing that the proposed empirical correlations perform reasonably well in soft to medium and even some sensitive clays as well as in uncemented sands with predominantly hard minerals. The reliability of the M vs. $E_D$ correlation in stiff and hard OC clays is at present less well documented [Boghrat (1982), Aas et al. (1984)]. The accumulating experience indicates that the M vs. $E_D$ correlation formulated by Marchetti (1980) represents an useful aid for the foundation designer when evaluating the compressibility of soils.
The future development of the DMT should also attempt to establish correlations E' vs. $E_D$ or G vs. $E_D$ especially for cohesionless deposits and which will be more strictly related to the calculation of deformations of geotechnical structures. The

two ways to approach this problem are to work out the proportionality coefficients from either $R_E = [E/E_D](1-\nu^2)$ or from $R_G = [G/E_D]2(1-\nu^3-\nu^2+\nu)$, using the appropriate laboratory and in situ tests. In the case of cohesive deposits the correlation of $E_D$ may be with E' as obtained from appropriate laboratory tests run on high quality undisturbed samples. In case of cohesionless soils, for which the availability of good undisturbed samples is still far from routine in everyday pratice, such correlations may be tentatively developed from the results of laboratory tests performed on reconstituted specimens or even more rationally from the results of other in situ tests which measure E or G directly [PLT, SBP, $CHV_s$, etc.].

Table XIII shows the $E_D$ obtained from DMT performed in the calibration chamber (CC) on the pluvially deposited dense Ticino sand [Baldi et al. (1981, 1983), Belotti et al. (1982, 1983)], which have already been mentioned in section 3.3.3. This table gives also the values of $E'_{25}$ and M as obtained for the same sand on pluvially-deposited specimens having equal relative densities to those in which DMT have been carried out.

Here:

- M = constrained modulus of the CC specimen. It corresponds in the case of NC specimens to the tangent modulus at the $\sigma'_v$ at which the DM has been inserted. In the case of OC specimens it corresponds to secant modulus of the unload curve made prior to insertion of the DM
- $E'_{25}$ = secant Young's modulus computed at one-fourth of the maximum deviator stress as obtained from $CK_oD$ triaxial compression tests [Bellotti et al. (1983)]
- $M_D$ = constrained modulus, function of $E_D$ and $K_D$ as obtained from Marchetti's (1980) correlation

The data collected in Table XIII allows the following remarks to be made:

1. For NC specimens the DMT of the Marchetti (1980) correlation has a tendency to underestimate the M value by an amount of 7% ± 26%. Two tests performed on the OC specimens of the same sand showed a similar trend:
2. The comparison between $E_D$ and $E'_{25}$ as obtained on pluvially deposited specimens of the same sand having the same $D_R$ and consolidation stress history leads to an $E'_{25}/E_D$ ratio for NC sand equal to 0.97 ± 0.20. Much higher $E_{25}/E_D$ ratios are obtained from OC specimens.

One recent CC DMT performed on a NC specimen of Norwegian Hokksund Sand having $D_R \cong 45\%$ showed the following results; $K_D = 2.27$; $E_D = 20.8$ MPa yielding $M_D = 23.9$ MPa. This value should be compared to the M determined during the consolidation phase of the CC which was equal to 40.8 MPa. The $E'_{25}$ of a pluvially deposited specimen of the same sand at the same $D_R$ and experiencing the same consolidation stresses as the CC specimen was equal to 19.8 MPa, yielding a $E'_{25}/E_D$ ratio similar to those obtained from Ticino Sand.

From a practical point of view it may be interesting to consider the possibility of correlation of $E_D$ vs G as obtained from the SBP or inferred from shear wave velocity measurements. The Table XIV shows the $G_{ur}$ as obtained from the SBP performed by TUT [Ghionna (1984)] in the Po River Valley in NC slightly silty medium dense to dense sand. These values may be compared to the $E_D$ values obtained in adjacent holes. Despite the scatter of the data which may be attributed to spatial soil variability and random testing errors, on average the proportionality coefficient $R_G$ is equal to 2.3 ± 0.5 for $\nu' = 0.3$. This indicates that for this sand deposit, the unload-reload SBP $G_{ur}$ is higher than the dilatometer shear modulus $G_D = E_D/2(1-\nu^3-\nu^2+\nu)$. Since both tests measure the soil deformability in the horizontal direction and reflect the "elastic" soil response, the observed difference can probably be attributed to the different degrees of disturbance caused by the insertion of the two instruments. This supposition is an agreement with Marchetti's (1982) opinion that the value of the proportionality coefficients like $R_M$, $R_E$, $R_G$, etc., depend primarily on the disturbance caused by the insertion of the dilatometer in the ground rather than on inherent differences existing between various types of moduli. In addition to what is stated the scatter in the $G_{ur}/E_D$ ratio may be due also to local soil variations, especially in the silt content, to which $E_D$ and $G_{ur}$ are quite sensitive.

TABLE XIII

Comparison Between the Dilatometer Modulus and Other Moduli of Ticino Sand as Obtained from CC Tests

| Test No. | OCR - | $D_R$ % | $\sigma'_{vp}$ kPa | $I_D$ | $K_D$ - | $E_D$ MPa | M MPa | $M_D$ MPa | $M/M_D$ | $E'_{25}$ MPa | $E'_{25}/E_D$ |
|---|---|---|---|---|---|---|---|---|---|---|---|
| 41 | 1 | 66.7 | 117 | 1.57 | 6.11 | 44.5 ± 154 | 78.5 | 90.9 | 0.864 | 31.6 | 0.71 |
| 42 | 1 | 77.6 | 321 | 1.30 | 4.55 | 77.6 ± 237 | 107.4 | 134.2 | 0.800 | 63.8 | 0.82 |
| 43 | 1 | 77.3 | 118 | 1.84 | 4.24 | 35.5 ± 148 | 77.7 | 59.9 | 1.285 | 40.0 | 1.13 |
| 44 | 1 | 79.3 | 221 | 1.59 | 4.36 | 54.9 ± 346 | 96.1 | 93.5 | 1.03 | 55.8 | 1.02 |
| 46 | 2.73 | 77.3 | 118 | 1.61 | 7.39 | 49.4 ± 281 | 127.1 | 109.1 | 1.165 | 94.7 | 1.92 |
| 47 | 1 | 77.6 | 66 | 2.40 | 4.33 | 25.9 ± 82 | 62.5 | 45.0 | 1.388 | 30.8 | 1.19 |
| 49 | 5.41 | 79.6 | 116 | 1.47 | 8.93 | 56.5 ± 208 | 194.4 | 135.0 | 1.440 | 212.6 | 3.78 |

$\sigma'_{vp}$ = effective vertical stress during penetration

TABLE XIV

Comparison Between $G_{ur}$ (SBP) and $E_D$ (DMT) as Obtained in Medium Dense to Dense Slightly Silty Sand of Po River Valley

| Hole No. | Depth m | $I_D$ - | $E_D$ MPa | $G_{ur}$ MPa | $G_{ur}/E_D$ |
|---|---|---|---|---|---|
| 1 | 7.3 | 2.38 | 52.6 | 30.2 | 0.57 |
| 1 | 8.8 | 2.82 | 35.8 | 36.8 | 1.03 |
| 1 | 10.3 | 2.36 | 35.8 | 32.7 | 0.91 |
| 1 | 11.8 | 2.56 | 30.7 | 30.2 | 0.98 |
| 1 | 13.3 | 2.21 | 64.9 | 40.9 | 0.63 |
| 1 | 16.3 | 1.98 | 32.2 | 42.5 | 1.32 |
| 1 | 17.8 | 1.78 | 54.7 | 51.1 | 0.93 |
| 3 | 6.2 | 1.82 | 31.6 | 25.6 | 0.81 |
| 3 | 7.7 | 1.68 | 18.6 | 24.6 | 1.32 |
| 3 | 9.2 | 1.25 | 36.7 | 31.1 | 0.85 |
| 3 | 10.7 | 1.64 | 39.5 | 40.7 | 1.03 |
| 3 | 12.2 | 1.81 | 47.6 | 50.0 | 1.05 |

From a review of experience to date in the use of DMT for assessing the soil deformation properties, the following remarks appear appropriate:

1. The correlation of M vs. $E_D$ as formulated by Marchetti (1980) seems to be reasonably accurate for the evaluation of the tangent constrained modulus in soft and medium clays at stress levels close to $\sigma'_{vo}$.
2. The same statement might apply to sands except the stress level to which $M = f(E_D, K_D)$ should be referred is not necessarily the same as $\sigma'_{vo}$. Different situations may be envisaged which depend on the hypothesis adopted concerning possible stress relaxation occurring between the insertion of the device and the evaluation of $p_o$ and $p_1$ [see Campanella and Robertson (1983), Campanella et al. (1985)].
3. For an evaluation of $M = f(E_D, K_D)$ in stiff and hard OC deposits, the available experimental evidence is rather scarce; therefore further research on the DMT as a tool for assessing M in such soils is needed.
4. Because of the simplicity, replicability, and low cost of the DMT, further research directed towards the determination of empirical correlations of $E_D$ vs. E' and G is advocated [Boghrat (1982), Campanella and Robertson (1983), Campanella et al. (1985)]. This step may be of particular practical interest for sands in which E' and G are more relevant for design than is M. The values of $G_{ur}$ determined from SBP and $G_{max}$ inferred from seismic in situ $V_s$ measurements might be suitable for correlation with $E_D$.
5. When using the soil deformation parameters determined by the DMT, the designer should focus attention on the following points [Marchetti (1980), Marchetti and Crapps (1981), Schmertmann (1983-a, 1984)]:
   - The dilatometer yields the soil modulus which, as intended by the developer, should correspond to the effective stress state existing in the ground prior to insertion of the device. This means that if the soil is even only lightly overconsolidated, the DMT will yield a reload modulus, which is not appropriate for evaluation of the soil deformations when the imposed loads induce stresses exceeding $\sigma'_p$.
   - Almost all existing experience in using the DMT for assessing M refer to soil deposits like marine and alluvial clays, silts, and predominantly quartz sands. Consequently the use of M from DMT run in "special" deposits from different geological environments such as residual soils, calcareous sands, loess, cemented and unsaturated soil, etc. should be done with caution.

3.3.5. Deformation Moduli from Plate Loading Tests (PLT)

The plate loading test (PLT) can be considered the earliest application of in situ testing for the evaluation of soil deformability. It is particularly useful in man made fills, in bouldery materials, hard fissured clays and soft rocks, i.e. in soils which generally cannot be adequately explored by other in situ techniques (with the exception of the geophysical methods).
The PLT is interpreted using formulae from the Theory of Elasticity. Usually the value of the shear modulus is computed assuming a rigid plate resting on an isotropic, homogeneous elastic half-space, or:

$$G = \frac{\Delta q}{s} \cdot \frac{\pi}{8} \cdot (1-\nu) \cdot f(z) \cdot B$$

where:

$\Delta q$ = net pressure increment for which G has to be computed
s = measured settlement
B = plate diameter
f(z) = correction factor considering the plate embedment, ranging from 1 at the surface to about 0.85 when plate rests on the bottom of an unlined exploratory shaft or borehole [Burland (1969)]

The PLT is well suited for exploration of cohesionless soils in which it is easy to achieve drained conditions. The use of PLT in cohesive deposits is generally limited to stiff and hard heavily OC fissured clays. In this case with a proper loading program and considering the high consolidation rates in reloading ($\Delta q + \sigma'_{vo} < \sigma'_p$), it is feasible to obtain drained soil moduli from the PLT.
In clays it is also possible to perform a "quick" PLT during which the plate is loaded in such a way to minimize the consolidation of the underlying soil. The moduli obtained from this type of test are generally considered as "almost undrained" and can be compared to those obtained from other supposedly undrained in situ and laboratory tests.
In clays the G from PLT is usually computed for the load interval between $q_1 = \sigma'_{vo}$ and $q_2 = 1/2\ q_{ult}$ (one-half of the ultimate bearing capacity) obtaining "some kind of $G_{50}$". In sands the G is evaluated in the range of load of practical interest, or between $\sigma'_{vo}$ and 0.2 or 0.25 of the estimated $q_{ult}$.
When comparing the secant G obtained from PLT to those resulting from other laboratory and in situ tests, the following aspects should be taken into consideration:

1. The G deduced from PLT reflects the behaviour of a relatively large volume of soil, including its macrofabric, fissures, etc. and therefore it cannot be compared directly to the stiffness of a macroelement obtained from laboratory tests [Marsland (1971, 1974)].

2. If one assumes that the soil behaviour can be more consistently represented by means of a cross-anisotropic elastic model [Gibson (1974)], then the PLT shear modulus should be considered a measure of shear deformation on the vertical planes $G_{vh}$ [Wroth et al. (1979)].

3. The PLT moduli are affected by the bedding errors arising from the preparation of the soil under the plate. This is especially true for tests in stiff fissured clays performed at the bottom of shafts and boreholes. In this case the most relevant factors are:
- the disturbance caused by drilling or excavation of the soil just below the plate;
- the time interval between drilling or excavation and the loading test, which may cause soil softening and allow fissures to open;
- the size of the plate and its ratio to the diameter of the borehole; the influence of the disturbed zone below the test plate on the obtained G decreases as B increases;
- the degree of restraint applied to the borehole or shaft.

Considerable positive experience in measuring the deformation moduli in stiff to hard heavily OC clays at the bottom of drilled boreholes has been gained in the last twenty years in the United Kingdom. Especially valuable are the experimental data obtained by Marsland and co-workers in London clay and in stiff glacial till [Marsland (1971, 1974)] from PLT's performed in unlined boreholes. These tests showed conclusively the poor reliability of the laboratory measured stiffness in these types of soils. A typical example is shown in Fig.70., which reports a comparison of G values obtained from laboratory, SBP and PL tests. The ratio between G (PLT) and G (Lab) varies between 1.8 and 4.8, and this is a typical range based on the UK experience. The G (PLT) appears to be consistent with those obtained from the back-analyses of the deformations measured in London clay around excavations and under foundations. When the aim is to perform PLT's at depth, screw plates are used since this type of test (SPLT) is faster and less expensive than the conventional PLT. This procedure, first developed about thirty years ago [Kummeneje (1956)], has received growing attention in the seventies [Schmertmann (1970), Janbu and Senneset (1973), Selvaduray and Nicholas (1979), Schwab and Broms (1977), Kay and Mitchell (1980), Berzins and Campanella (1981), Kay and Parry (1982)].
With the same theoretical assumptions adopted for the interpretation of the conventional PLT, the Young modulus E may calculated from the results of the SPLT from expressions of the following type:

$$E = A \frac{\Delta q}{s} B$$

where:
A = nondimensional factor depending on $\nu$ and the adhesion $c_a$ between plate and soil above it

Selvaduray and Nicholas (1979) indicated for undrained conditions ($\nu_u = 0.5$), A = 0.29 for $c_a/c_u = 1$ and A = 0.38 for a smooth plate, $c_a = 0$.
A different approach to interpret SPLT's in order to obtain M may be found in Janbu and Senneset (1973). They also report satisfactory predictions of the settlements of five buildings based on the results of SPLT.
Kay and Parry (1982) on the basis of SPLT performed in stiff Gault clay conclude that the screw plate is a valuable tool for determining both the undrained and drained moduli of stiff clays.

SPLT subjected to torsional loads applied through the rods has also been developed to assess the dynamic shear modulus at small strain levels [Schmertmann (1970), Bodare (1983)].
The writers believe that both the PLT and SPLT are useful tools for assessing the deformability of "difficult" soils in situ, especially in those deposits where either laboratory and other in situ tests are hardly reliable or even not feasible. However when using the soil moduli obtained from these tests, the designer must consider the following:

1. As all other in situ tests, the G (PLT) represents the average soil response from a large volume of soil which is difficult to associate with stress or strain level of the soil macroelement to which both the laboratory tests and the soil behavioural models are referred. Therefore $\Delta q$ for which the G from PLT is computed should at least approximately account for the stress or strain level which is anticipated for the project.

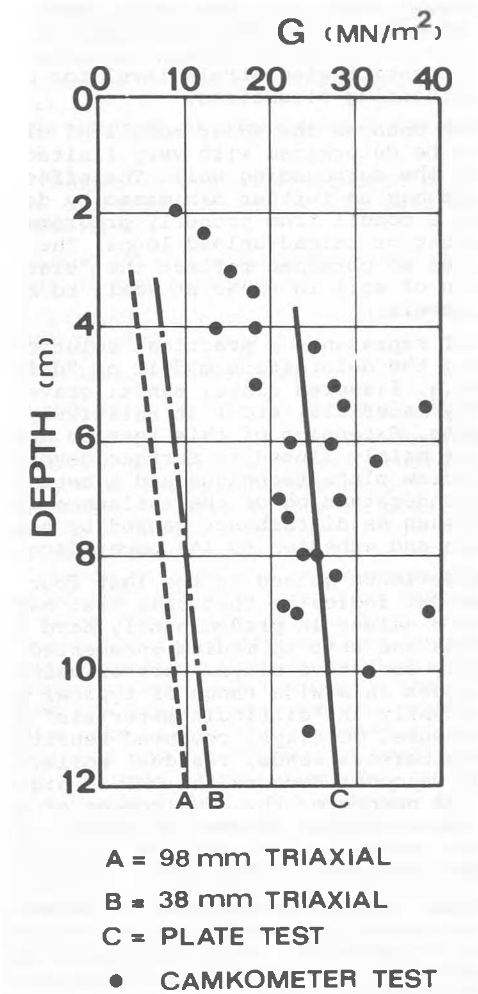



Fig.70: Values of Shear Moduli Measured by Various Tests in London Clay at Hendon Site. Data from Marsland (1971) and Windle (1976), Adapted by Wroth et al. (1979).

2. As mentioned in section 3.3.1, the soil deformation moduli depend on the current effective stress state in the soil below the plate. This leads to the fact that G below the plate is not constant, but varies with depth, reflecting the initial and final $\sigma'_v$, respectively, in undrained and drained tests. The obtained G reflects therefore an "average" stiffness of the soil between the elevation of the plate and depths of 1.5 B to 2 B (in dense and stiff soils) to 2 B to 3 B (in loose and soft soils). Therefore even in supposedly uniform deposits in which G and E increase with depth, the average modulus resulting from the PLT will depend also on the dimensions of the plate.

### 3.3.6. Summary and Conclusions.

1. The assessment of the soil moduli from in situ tests is of great practical interest. This is especially true in soil deposits such as highly fissured clays and cohesionless materials in which the possibility to obtain high quality undisturbed samples may be questionable. The full potential of in situ tests is partially hampered by the fact that they represent typical boundary value problems, which makes it difficult to connect the obtained moduli to either the strain level of the laboratory macroelement or to the anticipated strain level for different engineering situations.

2. The SBP enables the shear moduli of clays and sands to be determined with very limited disturbance of the surrounding soil. The effect of disturbance may be further minimized by determining the G moduli from properly programmed unload-reload or reload-unload loops. The $G_{ur}$ and $G_{ru}$ values so obtained reflect the "elastic" deformation of soil in shear at small to moderate strain levels.

3. The PLT represents a practical solution for assessing the deformation moduli of "difficult" soils (e.g. fissured clays, sands, gravels, boulderly materials, etc.) at relatively shallow depths. Extension of this test to greater depths is mainly linked to further developments of the screw plate technique and a better theoretical understanding of the influence of factors such as disturbance caused by plate penetration and adhesion to its upper face.

4. The experience gained in the last four years with the DMT indicates that this test may yield reliable M values in predominantly hard mineral sands and soft to medium uncemented and not highly sensitive clays. Further validation of this test in a wide range of typical soils and especially in "difficult materials" like stiff fissured OC clays, cemented sensitive clays, calcareous sands, residual soils, etc. would be welcome. Because the DMT is highly versatile and economic, the development of new empirical correlations between $E_D$ and E' or G, which are needed for deformation calculations especially in cohesionless soils, is encouraged.

5. Engineers frequently make use of correlations between $q_c$ and E' in sands. The writers believe that only a relatively weak link exists between these two parameters, since the $q_c$ is almost completely controlled by the shear strength and it is not very sensitive to the strain and stress level of sands.
On the basis of a comprehensive series of CC tests performed at different research institutions on predominantly quartz sands, the following points are clear:
   - in NC unaged uncemented sands, the commonly used $E'/q_c$ ratio varying between 2 and 3 appears to represent a reasonable engineering approximation;
   - in the same OC sands, the $E'/q_c$ ratio is between 3 to 6 times higher than for NC conditions.

## 3.4. A REVIEW OF IN SITU METHODS USED FOR DETERMINING FLOW AND CONSOLIDATION PROPERTIES

### 3.4.1. Introduction

Most problems in foundation engineering have to deal to an extent with the recognition and evaluation of some aspects of drainage, and in all of these aspects the permeability characteristics of soils have a controlling influence. Both for simplicity and in order to identify the type of problems we have to solve, it is usually accepted that in the broad distinction between coarse (gravel and sand) and fine grained (silt and clay) soils, the most obvious property difference is their permeability. No other geotechnical parameter has such a wide range and causes such completely different behaviour between coarse and fine grained soils. Coarse grained soils will act essentially as "open systems", with free drainage and fully effective shear strength; fine-grained soils ($k < 10^{-4}$ cm/sec) will act as "closed systems" during rapid application of stresses, with the development of pore pressures and changes of shearing strength during subsequent consolidation or swelling.
The large range of the coefficient of permeability may be explained by various factors controlling this parameter [Mitchell (1976)], the most important among them being the macro and microstructure of the deposit and, mainly in fine grained soil, the decrease in void ratio with increase of effective stress.
In particular, the dominating influence of the macrofabric of the deposit leads to large differences between the values of the flow and consolidation characteristics as derived from laboratory tests and those obtained from in situ tests and/or from back analysis of full scale structures or prototypes. In fact, the presence of thin layers and seams of more permeable material, as well as discontinuities in natural clay deposits, cause a local "anisotropy" with respect to permeability and a dependence of the laboratory-determined values of $c_v(c_h)$ and $k_v(k_h)$ on the specimen dimensions.
To a lesser extent the same conclusion applies to sands and coarse silts. Discrepancies between the field behaviour of monitored structures and laboratory test results can reach an order of magnitude, with the discrepancy increasing in nonuniform glacio-fluvial deposits because of erratic stratigraphy.
In view of these aspects, in many real cases the flow and consolidation properties can be evaluated only within an order of magnitude, but within this range a safe and economical design will often be possible [Milligan (1975), Mitchell et al. (1978)]. For many applications the primary objective is to determine if a problem exists. While the theory of water flow in porous media can be expressed simply and accurately, a reasonable determination of the in situ permeability is, in practice, difficult. Potential errors in insitu permeability measurements include [Mitchell et al. (1978)]:

. inaccurate water quantity measurement;
. inaccurate head measurement;
. insufficient test duration;
. inaccurate measurement of test section dimensions;
. plugging or smearing;
. hydraulic fracturing;
. unknown flow resistance of measuring system.

Not all of these errors can be easily isolated or mitigated. The methods presently in use are summarized in Table III. Except for the piezocone and the holding test performed by means of the self boring pressuremeter, the approach common to all other methods is to impose a known hydraulic gradient and measure the resulting flow.

Darcy's law provides the basis for the measurement of permeability, but deviations from direct proportionality between flow velocity and hydraulic gradient may sometimes occur if the soil structure changes during flow, e.g., in coarse-grained soils, or under very small hydraulic gradients in clays [Hansbo (1960)]. Other relevant aspects of the various methods are summarized in the remainder of this section.

3.4.2. Pumping Tests

Borehole permeability tests are performed by pumping water into (infiltration or outflow test) or out of (drawdown or inflow test) a borehole. Generally, infiltration tests performed at a constant head are preferred because it is easier to measure the water level in the borehole, and to produce a drawdown during constant pumping, the use of expensive pumps may be required.

Formulas to obtain the permeability coefficient are reported by Hvorslev (1951) and Cassan (1980) for different test section geometries and boundary conditions. Generally the following theoretical assumptions are made:

- the hydraulic head is assumed constant in all points of the test section;
- the problem is axisymmetric.

All these assumptions may be accepted in practice when the test is performed below the phreatic surface, but above the water table the first assumption no longer applies. This extremely complex problem can however be treated using solutions by Ivakir (1947), Nasberg (1951) and Cassan (1980). According to Mitchell et al. (1978) and Milligan (1975) the main advantage of the borehole permeability test is its simplicity. Because of the relatively small volume of soil tested, the results obtained may be misleading in erratic soils. However even in homogeneous deposits they have been thought to be only within an order of magnitude of the in situ permeability. Moreover, outflow tests may be affected by the clogging of the filter with dislodged particles, so that the resulting $k_{out}$ permeability values is lower than the in situ value. Inflow tests may cause erosion of particles from the surrounding soil, resulting in a higher $k_{in}$ value. To check these potential sources of errors, it may be convenient to perform tests under several different heads. The best but also the most expensive method presently available to estimate the permeability in a relatively pervious deposit ($k > 10^{-4}$ cm/sec) is a large scale pumping test. Such tests are performed on a well installed with a low loss screen and filter gravel pack and which, if possible, fully penetrates the acquifer to be tested. The drawdown in the acquifer should be monitored with four or six piezometers along two lines perpendicular to each other and passing through the well. The permeability coefficient is obtained from solutions available in the literature, with due consideration to whether the test is a "steady state" or a "transient flow" test, whether the well is fully or partially penetrating the acquifer, and whether the acquifer is an artesian or a gravity one [Thiens (1906), Dupuit (1963), Mansur and Kaufman (1962), Theis (1935)].

Taking into account the soil anisotropy, the obtained value of the coefficient of permeability should be considered representative of horizontal flow $k_h$ for a fully penetrating well and an average value $k_{av}$ when a partially penetrating well is used.

3.4.3. Tests in Piezometers

Due to the low permeability ($k < 10^{-4}$ cm/sec) of fine-grained soils, large scale tests cannot be used in these materials because they are time consuming. Instead piezometers have been employed in somes sites, either installed in predrilled boreholes [Casagrande (1946)] or pushed below the bottom of a borehole [Wilkes (1970), Parry (1971)] in order to provide data.

In general, compared to coarse-grained soils, tests in fine-grained soils are difficult to perform and even more difficult to interpret due to changes in the imposed effective stresses which cause changes in the consolidation and flow parameters.

The main sources of experimental errors are related to:

1. The danger of hydraulic fracturing, due both to the installation and to the head applied during the test.
2. Smearing of the surrounding soil during the pushing of the piezometer.
3. The permeability of the porous element with respect to that of the soil.
4. The hydraulic time-lag.
5. The presence of gas in the pore water.

To avoid the danger of hydraulic fracturing constant head tests are preferred (there are also advantages in their interpretation) with applied excess pore pressures as low as possible but sufficient to obtain a reasonable flow rate.

To minimize "smear" effects it is suggested that the distance to push the piezometer below the bottom of the borehole should be restricted to about twice the length of the tip, keeping the piezometer saturated during the insertion.

When applying the hydraulic head difference, consolidation or swelling occurs in the soil surrounding the piezometer, and the measuring of the flow rate allows the computation of both the permeability and consolidation coefficient, according to basic theory developed by Hvorslev (1951), Gibson (1963), and extended by Wilkinson (1967, 1968) and Jezequel and Mieussens (1975)].

In a constant head test, where the flow is directed from the piezometer towards the soil (outflow test), the obtained coefficients of permeability and consolidation are representative for unload-reload conditions. These values reflect the soil behaviour in the OC state and tend to be appreciably higher than those obtainable for primary loading corresponding to NC conditions. The use of the k and c values obtained from outflow tests in design is, therefore, strictly speaking appropriate only when the condition $\Delta\sigma_v + \sigma'_{vo} < \sigma'_p$ is fullfilled, where $\Delta\sigma_v$ is the imposed increment

of vertical stress.
When the examined field problem leads to a situation where the $\sigma'_{vo} + \Delta\sigma_v > \sigma'_p$, the flow and consolidation characteristics must be determined from inflow tests, with the flow directed from the soil towards the piezometers.

### 3.4.4. Self-Boring Cells

The problems of soil disturbance and smear related to the installation of piezometers may be overcome at least to a great extent by using equipment operating on the self-boring principle. With the self-boring permeameter [Jezequel and Mieussens (1975)], it is possible to determine $k_h$ and $c_h$ by constant head permeability tests. An example of the measurement of $k_h$ by means of this device is reported in Fig.71 for Porto Tolle silty clay [Jamiolkowski et al. (1980)].

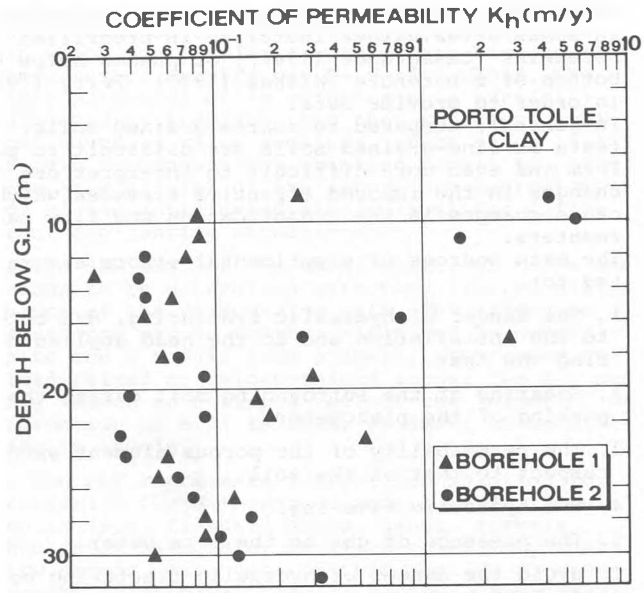



Fig.71: Example of Coefficient of Permeability Determined from Self-Boring Permeameter Tests [Jamiolkowski et al. (1980)].

The large scatter of the measured values is attributed to the presence of a highly developed macro-fabric, see Fig.36. These values have been combined with deformability from oedometer tests to get the $c_h$ values shown in Table XX (see section 3.4.5.). When making comparisons with $c_h$ values from other in situ tests or from back-analysis of full-scale structures, it must be taken into account that the apparatus is presently set up to allow only tests with outward flow to be performed. Therefore, due to the reasons previously discussed, it is advisable to perform the tests under only small values of applied hydraulic head. Even with this precaution, $k_h$ and $c_h$ are determined in a stress range near or slightly below $\sigma'_{vo}$, thus are still not representative for NC conditions. The result is better suited to solve field problems for which the condition $\Delta\sigma_v + \sigma'_{vo} \leq \sigma'_p$ exists.
Another promising example of the application of the self-boring technique for the determination of $c_h$ has been presented by Clarke et al. (1979). These authors using the SBP - Camkometer determined $c_h$ by means of so-called "holding tests" in which the probe was expanded to a predetermined value of cavity strain (e.g., 6 to 10%) and thereafter the applied cavity pressure was adjusted in order to keep the radius of the expanded cavity constant with time. The decay of the excess pore pressure induced by the expansion of the probe was monitored by two pore pressure tranducers located at mid-height of the cavity wall which permitted the determination of an entire dissipation curve.
The interpretation of holding tests is based on the following assumptions:

1. The initial distribution of the excess pore pressure is that resulting from the expansion of an infinitely long cylindrical cavity in an elastic perfectly-plastic medium under the condition of no volume change [see Clarke et al. (1979)].

2. The consolidation process around the expanded probe is analyzed with reference to the closed form solution proposed by Randolph and Wroth (1978). These authors give the basic equations which couple the linear-elastic stress-strain response of the soil skeleton to the radial excess pore water flow, assuming its initial logarithmic distribution radially from the probe face to zero at the outer limit of the plastic zone.

The solution of these equations leads to the theoretical variation of $\Delta u$ as a function of time at the probe-soil interface, as shown in Fig. 72(a).
On the basis of this solution, the evaluation of the $c_h$ from a holding test can be done as follows:

1. Theoretical generated $\Delta u_{max}$ at the cavity wall, which is then the reference initial $\Delta u$, is evaluated from the following formula:

$$\Delta u_{max} = c_u \ \ln \frac{G}{c_u} \ \frac{\Delta V}{V_c}$$

where:
$G$, $c_u$ = respectively, shear modulus and undrained strength evaluated from the loading portion of the SBP
$\Delta V/V_c$ = volumetric cavity strain at the start of the holding test

2. The observed time necessary to achieve a preselected percent of the $\Delta u_{max}$ is then computed, assuming as the zero reference the time at which the maximum excess pore pressure has been observed. Usually reference is made to the time for 50% dissipation of the excess pore pressure. In this case the time factor $T_{50}$ may be computed directly from Fig.72(b).

3. The $c_h$ is thus computed from equation:

$$c_h = \frac{T\ r_c^2}{t}$$

where:
$r_c$ = current cavity radius at which holding test is performed

4. The value of $k_h$ may then be inferred from the following relation:

$$c_h = \frac{k_h}{\gamma_w} \quad 2\ G\ \frac{1 - \nu'}{1 - 2\nu'}$$

where:
$\gamma_w$ = the density of the pore fluid

The results of successfully performed holding tests may be found in Clarke et al. (1979), Clarke

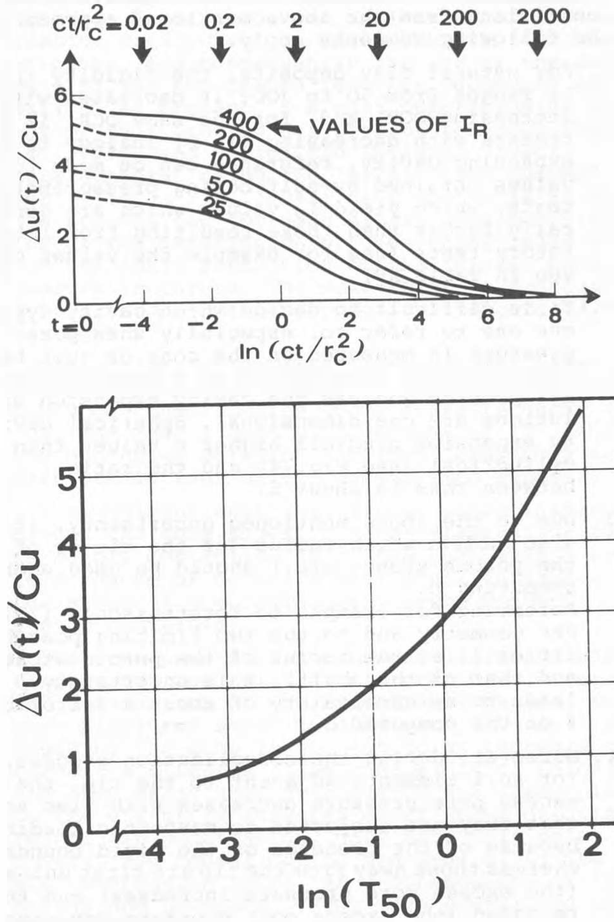



Fig.72: (a) Variation of Excess Pore Pressure as a Function of Time Factor [Randolph and Wroth (1978)]. (b) Time Factor for 50% dissipation of excess Pore Pressure [Clarke et al. (1979)].

(1981) and Benoit (1983).
The $c_h$ obtained by these authors from the holding tests when compared to the laboratory determined $c_v$ values indicated the following:

Canvey Island, soft organic silty clay:

$$\frac{c_h}{c_v} = 20 \text{ to } 305$$

San Francisco Bay Mud, soft silty clay:

$$\frac{c_h}{c_v} = 11 \text{ to } 16$$

### 3.4.5. Piezocone Dissipation Tests

During the last ten years, much attention has been devoted to investigating the usefulness and the meaning of dissipation tests in clays performed with the piezocone (CPTU) or the pore pressure probe (PPP) (see Fig.30). The tests consist of stopping the steady penetration of the cone tip and monitoring the decrease of the excess pore pressure with time (Fig.73), from which an approximate value of the coefficient of consolidation can be obtained.

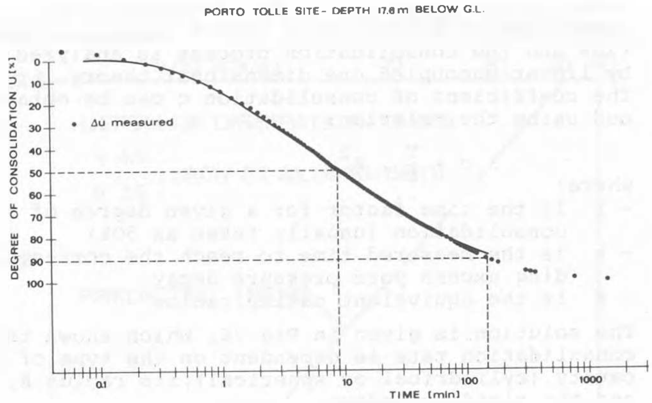



Fig.73: Example of Dissipation Test from Piezocone. [Battaglio et al. (1981)].

This test provides a cost-effective tool for obtaining reproducible information about the variation with depth of flow and consolidation soil characteristics.
In spite of the advantages of the test, dissipation records are very difficult to interpret because of [Baligh and Levadoux (1980)]:

1. Uncertainties due to the initial excess pore pressure distribution around the cone, which arises from the two dimensional nature of the steady cone penetration process.
2. Complex soil behaviour, requiring consideration of anisotropy and nonlinearity.
3. Two-dimensional process of consolidation around the cone tip and the high hydraulic gradients associated with the initial excess pore pressure distribution.
4. Coupling between total stresses and pore pressures during consolidation.
5. Uncertainties in determining the level of shearing during consolidation.

Due to these difficulties, important aspects required for the interpretation of the piezocone dissipation tests are still far from being solved satisfactorily. Among them, the following are of immediate practical interest:

1. Where should the pore pressure be measured, on the cone or behind it, in reference to test repeatability and to the theories used in its interpretation?
2. What degree of consolidation should be achieved in order to obtain significant properties information, but also to have a test economical to perform in practice?
3. What coefficient of consolidation (vertical, horizontal, average) is measured?
4. What is the meaning of the measured coefficient of consolidation, i.e. what practical problems is it applicable to, considering the stress level and soil nonlinearity?

A simplified approach has been proposed by Torstensson (1975, 1977) for interpretation of piezocone dissipation records.
The soil is assumed to behave as an elastic-perfectly plastic material subjected to isotropic initial stress. The initial excess pore pressure

distribution is estimated by one-dimensional (spherical or cylindrical) cavity expansion theories and the consolidation process is analyzed by linear uncoupled one dimensional theory.
The coefficient of consolidation c can be obtained using the relation:

$$c = \frac{T}{t} \cdot R^2$$

where:
- T is the time factor for a given degree of consolidation (usually taken as 50%)
- t is the measured time to reach the corresponding excess pore pressure decay
- R is the equivalent cavity radius

The solution is given in Fig.74, which shows that consolidation rate is dependent on the type of cavity (cylindrical or spherical), its radius R, and the rigidity index

$$I_R = \frac{G_u}{c_u}$$

where:
$G_u$ = undrained shear modulus, usually assumed equal to $G_{u50}$ which is the secant modulus at half the applied principal stress difference at failure
$c_u$ = undrained shear strength of the clay

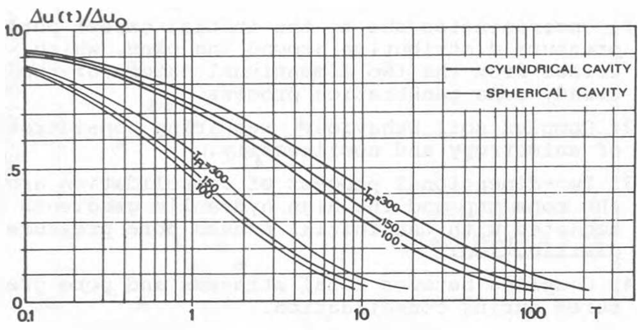



Fig.74: Pore Pressure Decay vs. Time Factor According to Torstensson (1975).

Considering an estimate of the consolidation coefficient from the above mentioned approach, the following comments apply.

1. For natural clay deposits, the rigidity index $I_R$ ranges from 50 to 300; it decreases with increasing OCR, and, for the same OCR, it increases with decreasing PI. By analogy to the expanding cavity, reference can be made to values obtained by self-boring pressuremeter tests, which yield $I_R$ values which are generally larger than those resulting from laboratory tests (see for example the values given in Table XV).
2. It is difficult to decide which cavity type one has to refer to, especially when pore pressure is measured on the cone or just behind it. Cone penetration is a two-dimensional problem whereas the cavity expansion solutions are one-dimensional. Spherical cavity expansion predicts higher c values than cylindrical (see Fig.74) and the ratio between them is about 5.
3. Due to the above mentioned uncertainty, it is also unclear which radius (of the tip or of the porous stone, etc.) should be used when computing c.
Referring for example to Torstensson's (1975) PPP geometry and to the two limiting possibilities (i.e. the radius of the porous stone and that of the shaft), this uncertainty leads to an uncertainty of about a factor of 4 on the computed c.
4. Moreover, during the consolidation process, for soil elements adjacent to the tip, the excess pore pressure decreases with time so that they are subjected to monotonic loading because of the presence of the rigid boundary, whereas those away from the tip are first unloaded (the excess pore pressure increases) and then reloaded (the excess pore pressure decreases) [see Carter et al. (1978)]. This leads to the conclusion that the consolidation coefficient yielded by the test should not be considered as an unique value, so it is not clear what is the meaning of the c value predicted according to the Torstensson approach.

TABLE XV

Rigidity Index of Some Soft Natural Clay Deposits [Ghionna et al. (1981)]

| Site | | PI (%) | OCR | Rigidity Index $I_R = G_{u50} / c_u$ | | $S_t$ |
|---|---|---|---|---|---|---|
| | | | | $CK_oU$-DSS | SBP *** | |
| Trieste | (Italy) | 47 ± 10 | 1 | 170 | 206 | 3 to 4 |
| Porto Tolle | (Italy) | 31 ± 2 | 1 | 110 | 290 | 2 to 3 |
| Panigaglia | (Italy) | 45 to 65 | 1 to 1.2 | 135 | 478 | 3 to 5 |
| Drammen | (Norway) | 25 to 30 | 1.2 to 1.5 | 203* | 184 | 7 to 8 |
| Onsöy | (Norway) | 20 to 35 | 1.2 to 1.7 | 163** | 365 | 5 to 6 |
| Bandar Abbas | (Iran) | 33 ± 6 | 1.5 to 2 | NA | 133 | 2 to 4 |

* upper plastic clay
** appropriate for the lower OCR values
*** $G_{u50}$, $c_u$ determined on the stress strain curve as derived from SBP tests

$S_t$ = sensitivity from field vane tests
NA = not available

In view of this brief summary, interpretation of piezocone dissipation for evaluation of the consolidation coefficient is still subject to much uncertainty, and consequently the use of the obtained c values for design requires judgement and caution. At present, the must comprehensive treatment of this problem has been by Levadoux and Baligh (1980) and Baligh and Levadoux (1980). These authors, using the "strain path method" [see Baligh (1972, 1984)] have been able to take into account the two-dimensional, axisymmetric nature of the penetration process which leads to a more realistic prediction of the initial pore pressure isochrone. The most relevant conclusions, of both theoretical and practical importance of this work are:

1. The effect of coupling between total stresses and pore pressure is small, except at the early state of consolidation (pore pressure decay $\leq$ 20%) and near a cone with an apex of 18°; thus the uncoupled solution provided reasonably accurate predictions of the dissipation process.
2. A two-dimensional analysis of consolidation around the cone shows that the dissipation rate is mainly controlled by $c_h$ and that even a tenfold change of $c_v$ has a negligible influence on the shape of the isochrones; hence the test yields $c_h$ values.
3. The cone penetration produces undrained shearing of the soil with a pore pressure increase and a reduction of the effective stresses. When pore pressures start to dissipate, the soil surrounding the cone is subject to an increase of effective stresses under conditions of reloading, and only after some dissipation has taken place do the effective stresses equal those existing before tip penetration. Only from this point onward, the consolidation proceeds along the virgin compression curve. As a consequence of this, Baligh and Levadoux (1980) have postulated that $c_h$ obtained from the early stage of dissipation (less than 50% of consolidation) is relevant for reloading conditions reflecting therefore behaviour of OC soils.
4. For the investigated locations of the porous stone (Fig.75), Baligh and Levadoux (1980) showed that in NC and slightly OC Boston Blue clay:
   - For an 18° cone the dissipation rate is influenced by the position of the filter.
   - For a 60° cone, the filter located on the cone face or at its base yields substantially equal dissipation rates.
   - For filter stones located far behind the cone, the dissipation rate is no longer influenced by the tip geometry but the rate is appreciably slower.

Based on these findings, Baligh and Levadoux (1980) recommended the following tentative procedure for the evaluation $c_h$ from the CPTU dissipation tests:

1. The normalized pore pressure excess

$$\frac{\Delta u\ (t)}{\Delta u_o}$$

where:
$\Delta u_o$ = initial excess pore pressure
$\Delta t\ (t)$ = excess pore pressure at a given time t,
should be plotted versus the time factor T (log scale) (see Fig.76).

2. The measured dissipation curves should be homothetic with respect to the corresponding theoretical ones. Infact, according to the linear consolidation theory, a horizontal tran-

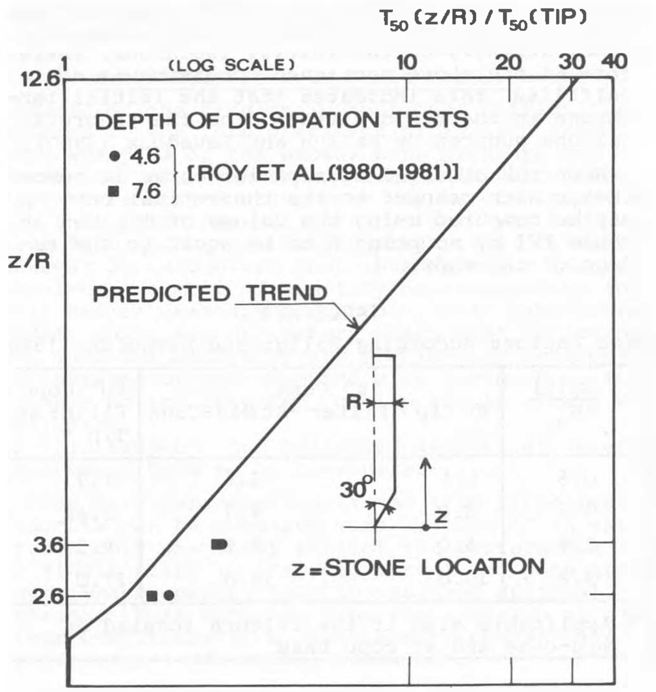



Fig.75: Predicted [Baligh and Levadoux (1980)] vs. Measured [Roy et al. (1980, 1981)] Dissipation Rates for Different Porous Stone Locations.

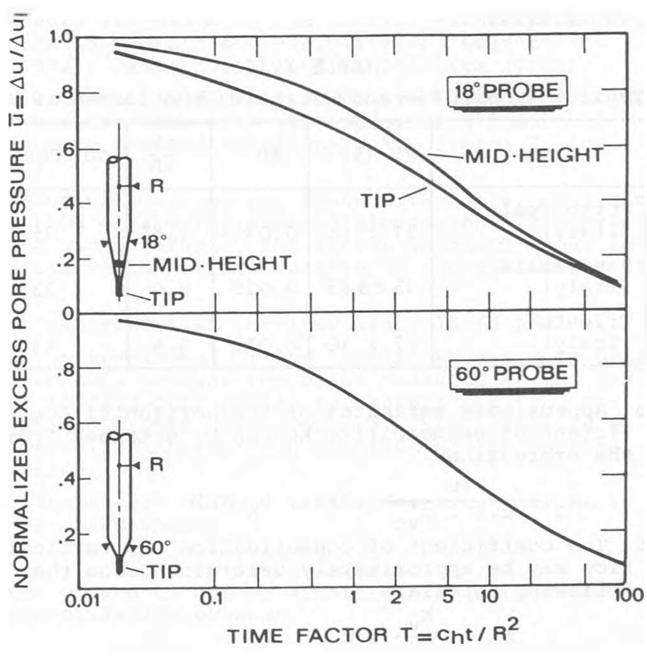



Fig.76: Pore Pressure Decay vs. Time Factor from Strain Path Method [Baligh and Levadoux (1980)].

slation of the dissipation curves reflects only changes in $c_h$, whereas their overall shape depends strongly on the initial isochrone. Therefore if the above mentioned condition is not fulfilled this indicates that the initial isochrone in the field differs from the theoretical one adopted by Baligh and Levadoux (1980).

3. When the observed dissipation curve is homothetic with respect to the theoretical one, $c_h$ may be computed using the values of T given in Table XVI by adopting R to be equal to the radius of the rods.

TABLE XVI

Time Factors According Baligh and Levadoux (1980)

| $\frac{\Delta u(t)}{\Delta u_o}$ | 18° Cone At tip ← Filter | 18° Cone →At Mid-Cone | 60° Cone Filter at Tip * |
|---|---|---|---|
| 0.6 | 1.4 | 2.6 | 1.9 |
| 0.5 | 3.0 | 4.7 | 3.65 |
| 0.4 | 6.0 | 8.2 | 6.5 |
| 0.2 | 30.0 | 34.0 | 27.0 |

* Applicable also to the filters located at mid-cone and at cone base

4. The value of $c_h$ evaluated for $\Delta u(t)/\Delta u_o = 0.5$ may be used in problems involving horizontal flow in the OC range. In order to obtain the coefficient of consolidation in the NC range, the following rule is suggested:

$$c_h\ (NC) = \frac{RR}{CR} \cdot c_h\ (OC)$$

where:
RR = recompression ratio
CR = virgin compression ratio
For some typical values of the ratio CR/RR, see Table XVII.

TABLE XVII

Typical Values of RR and the ratio CR/RR for Soft Clays

| Site | PI (%) | RR | $\frac{CR}{RR}$ | No. Tests |
|---|---|---|---|---|
| Porto Tolle (Italy) | 31 ± 12 | 0.031 | 6.5 | 95 |
| Panigaglia (Italy) | 45 to 65 | 0.025 | 8.0 | 33 |
| Trieste (Italy) | 47 ± 10 | 0.032 | 6.6 | 43 |

5. Approximate estimates of the horizontal coefficient of permeability $k_h$ can be obtained from the expression:

$$k_h = \frac{\gamma_w}{2.3 \cdot \sigma'_{vo}} \cdot RR \cdot c_h\ (OC)$$

6. The coefficient of consolidation for vertical flow may be approximately determined from the following formula:

$$c_v = \frac{k_v}{k_h} \cdot c_h$$

A rough estimate of the in situ anisotropy of the permeability of clays, as expressed by the ratio $k_h/k_v$, can be made on the basis of the data given in Table XVIII.

TABLE XVIII

Range of Possible Field Values of the ratio $k_h/k_v$ for Soft Clays

| Nature of Clay | $k_h$ / $k_v$ |
|---|---|
| - no macrofabric, or only slightly developed macrofabric, essentially homogeneous deposits | 1 to 1.5 |
| - from fairly well to well developed macrofabric, e.g. sedimentary clays with discontinuous lenses and layers of more permeable material | 2 to 4 |
| - varved clays and other deposits containing embedded and more or less continuous permeable layers | 3 to 15 |

The above analysis of CPTU dissipation tests by Baligh and Levadoux (1980) does not include any possible influence of smear generated by the penetration process, which may lead, if not taken into consideration, to an underestimate of the computed $c_h$. The numerical parametric analyses made to clarify this aspect of the problem [Acar et al. (1982), Fioravanti (1983)] lead however to the conclusion that in virtue of all the uncertainties presently incorporated into the interpretation of CPTU dissipation records, the influence of smear may be relevant only in sensitive soils with a highly pronounced permeability anisotropy. In that extreme condition, if one assumes a fivefold reduction of the permeability in the smeared zone whose radial extension is 1.5 R, neglecting its presence will cause the computed $c_h$ to be underestimated by a factor ranging between 2 and 3. In low sensitivity cohesive deposits this factor would be much lower, ranging between 1.3 and 1.5.
Finally the writers want to stress the fact that apart from the theoretical aspects of the analysis of CPTU dissipation records, the most important condition for their successful interpretation and use in practice is to a large extent controlled by the requirement of a rigid and extremely well de-aired pore pressure measuring system [Gillespie and Campanella (1981)].

As an example, Table XIX reports the values of $c_h$ obtained by Battaglio and Maniscalco (1983) at Porto Tolle site using both the Torstensson and the Wissa type probes. These results are compared in Table XX with those resulting from back analysis of the behaviour of a full scale preloading embankment [Jamiolkowski and Lancellotta (1984)]. The following comments apply to these data:

1. The values obtained from piezometer readings refer only to pore pressure readings during the loading stage (see also section 3.4.6). Because (1) the stress history of the deposit (see Fig.60), (2) the predicted initial excess pore pressure value which is 1.17 times the applied $\Delta\sigma_{vo}$, and (3) the degree of consolidation on the basis of settlement records which is from 20% to 30% during the loading stage, the values in Table XX are representative of soil condition where $\sigma'_{vo} + \Delta\sigma'_v < \sigma'_p$; hence they should be compared with the $c_h(OC)$ probe values.

TABLE XIX

Values of $t_{50}$ and $c_h$ obtained at Porto Tolle Site with the Piezocone [Battaglio and Maniscalco (1983)]

| Depth m | $t_{50}$ sec | $c_h$ (OC) $m^2/y$ | $c_h$ (NC) $m^2/y$ | Notes |
|---|---|---|---|---|
| 10.70 | 393 | 77 | 12 | T |
| 11.00 | 210 | 179 | 28 | W |
| 11.40 | 312 | 119 | 18 | W |
| 12.40 | 240 | 149 | 23 | W |
| 12.50 | 275 | 110 | 17 | T |
| 13.50 | 354 | 86 | 13 | T |
| 14.60 | 284 | 107 | 16 | T |
| 14.80 | 360 | 99 | 15 | W |
| 17.40 | 256 | 118 | 18 | T |
| 17.60 | 533 | 57 | 9 | T |
| 19.80 | 132 | 278 | 43 | W |
| 20.50 | 344 | 88 | 14 | T |
| 21.80 | 300 | 119 | 18 | W |
| 22.90 | 169 | 179 | 28 | T |
| 23.80 | 660 | 60 | 9 | W |
| 28.90 | 300 | 119 | 18 | W |

T = Torstensson's PPP
W = Wissa's CPTU
Note: the $c_h$ values have been obtained using the Baligh and Levadoux (1980) procedure

TABLE XX

Comparison Between $c_h$ values from Piezocone Dissipation Tests and Values from Back-Analysis of a Full Scale Preloading Embankment

| | $c_h$ (OC) $m^2/y$ | $c_h$ (NC) $m^2/y$ | References |
|---|---|---|---|
| Piezocone | 122 ± 55 | 19 ± 9 | Battaglio and Maniscalco (1983) |
| Back-analysis of piezometer readings | 63 - 95 | - | Jamiolkowski and Lancellotta (1984) |
| Back-analysis of settlement records | - | 15 | |

N.B.: Laboratory values = 9-17 $m^2/y$
Field permeability (self-boring permeameter) times lab. $m_v$ = 22-32 $m^2/y$

2. The values from settlement records refer instead to the truly NC range, so that they can be compared with the $c_h$(NC) values obtained with the piezocone.

3. The scatter in the piezocone data reflects the highly developed macrofabric of this deposit (see Fig.36).

4. The data from pore pressure readings and settlement records are from the embankment section on jetted sand drains, for which smear effects presumably do not exist [Jamiolkowski and Lancellotta (1984)].

3.4.6. Back-Analysis from Field Measurements

Because of the previously mentioned difficulties related to the interpretation of in situ tests, back-analysis of the excess pore pressure and strains or displacements measured in full-scale structures or/and prototypes may be considered at least in principle the most reliable way to assess soil flow and consolidation properties. It must however be recognized that this approach always involves a number of simplifying assumptions (boundary and/or drainage conditions, soil behavioural model, auxiliary parameters which must be introduced into the analysis, etc.) which may enhance the reliability of the computed c or k and whose influence on the obtained results should always be evaluated.
In this respect, the following aspects of general importance have to be considered:

1. The soil parameters obtained from field measurements can be compared with results of in situ tests only when they reflect the performance at a single point in the soil mass (i.e. pore pressure measurements, locally measured strains), whereas those obtained from the analysis of average conditions (i.e. settlement records) are representative of overall behaviour.

2. The methodology by which the soil parameters are determined depends on the way they are going to be used, i.e. the parameters and the methods of analysis are strictly related [Lambe, 1973)].

With these points in mind, in the following section the assumptions and errors of field measurements are discussed.
The procedure for evaluating the coefficient of consolidation ($c_v$ or $c_h$) from pore pressure records is as follows:

1. Determine from observation or compute from theory the value of the initial excess pore pressure $\Delta u_o$ [Wroth and Parry (1977), Ladd et al. (1972), Jamiolkowski and Lancellotta (1984)].

2. Determine ratios of $\Delta u/\Delta u_o$ for different times t and use them with the appropriate theory to compute the nondimensional time factor $T_v$ (or $T_h$).

3. Evaluate $c_v$ (or $c_h$) using either the total time method or the incremental time method [Bromwell and Lambe (1968)]. The latter approach tends to minimize the errors related to the estimation of $\Delta u_o$.

The uncertainties involved are related to:

1. The assessment of the initial excess pore pressure $\Delta u_o$, because the value measured at the end of loading only rarely represents the true undrained excess pore pressure. This aspect is especially important in the presence of vertical drains.

2. The stress history of the deposit, because in all cases where:

$$\sigma'_{vo} < \sigma'_p < \sigma'_{vo} + \Delta\sigma_v$$

the $c_v$ and $c_h$ change significantly during the consolidation process.

3. The location of the piezometer with respect to that of the drain when vertical drains are used for speeding up consolidation. In this respect, very little experimental data are available about the deviations from the vertical of both drains and

piezometers.
These deviations may reach the following values [Jamiolkowski et al. (1983)]:

- driven displacement drains ($0.3 \leq d_w \leq 0.6$ m): 0.01 to 0.02;
- prefabricated drains of small dimensions: up to 0.025;
- piezometers installed in boreholes: 0.01 (this value becomes larger with pushed-in piezometers).

An example of the difficulties involved in such analyses, mainly in evaluating the initial excess pore pressure response $\Delta u_o$, is given by Jamiolkowski and Lancellotta (1984). They examined the behaviour of a trial embankment on four different types of vertical drains (see Fig.77). Four different approaches for predicted $\Delta u_o$ were compared [see also Foott and Ladd (1973)]:

1. Assume empirically that $\Delta u_o = \Delta\sigma_v$.
2. Assume that the soil behaves as an elastic-perfectly plastic material governed by the Tresca failure criterion. In this case, as long as the soil around the piezometer responds "elastically", $\Delta u_o = \Delta\sigma_{oct}$; when a condition of contained plastic flow is reached, then $\Delta u_o = \Delta\sigma_v$.
3. Assume semi-empirically that $\Delta u_o$ can be predicted by pore pressure coefficients [Skempton (1954), Henkel (1960)].
4. Assume that the soil behaviour is in agreement with the modified Cam Clay model (MCCM); see Burland (1967) and Wroth and Parry (1977).

Results of these four approaches applied to this case record are given in Table XXI and the following comments can be made:

TABLE XXI

Predicted vs. Measured Excess Pore Pressure of Piezometer EP2 ($\Delta u_{max}$ = excess pore pressure measured at the end of the loading ramp) [Jamiolkowski and Lancellotta (1984)]

| | Approach | | | |
|---|---|---|---|---|
| | 1. $\Delta u_o = \Delta\sigma_v$ | 2. Elasto-Plastic | 3. Pore Pressure Coefficient | 4. Cam Clay |
| $\frac{\Delta u_{max}}{\Delta u_o}$ | 0.91 | 1.11 | 0.89 | 0.78 |
| $\frac{\Delta u_o}{\Delta\sigma_v}$ | 1 | 0.82 | 1.03 | 1.17 |

1. The elasto-plastic approach leads to unrealistic results; it predicts $\Delta u_o$ which is lower than or equal to that measured at the end of the loading ramp $\Delta u_{max}$. This is particularly unsatisfactory in the presence of vertical drains.
2. The empirical and semi-empirical approaches (1 and 3) yield more realistic comparisons between $\Delta u_o$ and $\Delta u_{max}$, but they still lead to a

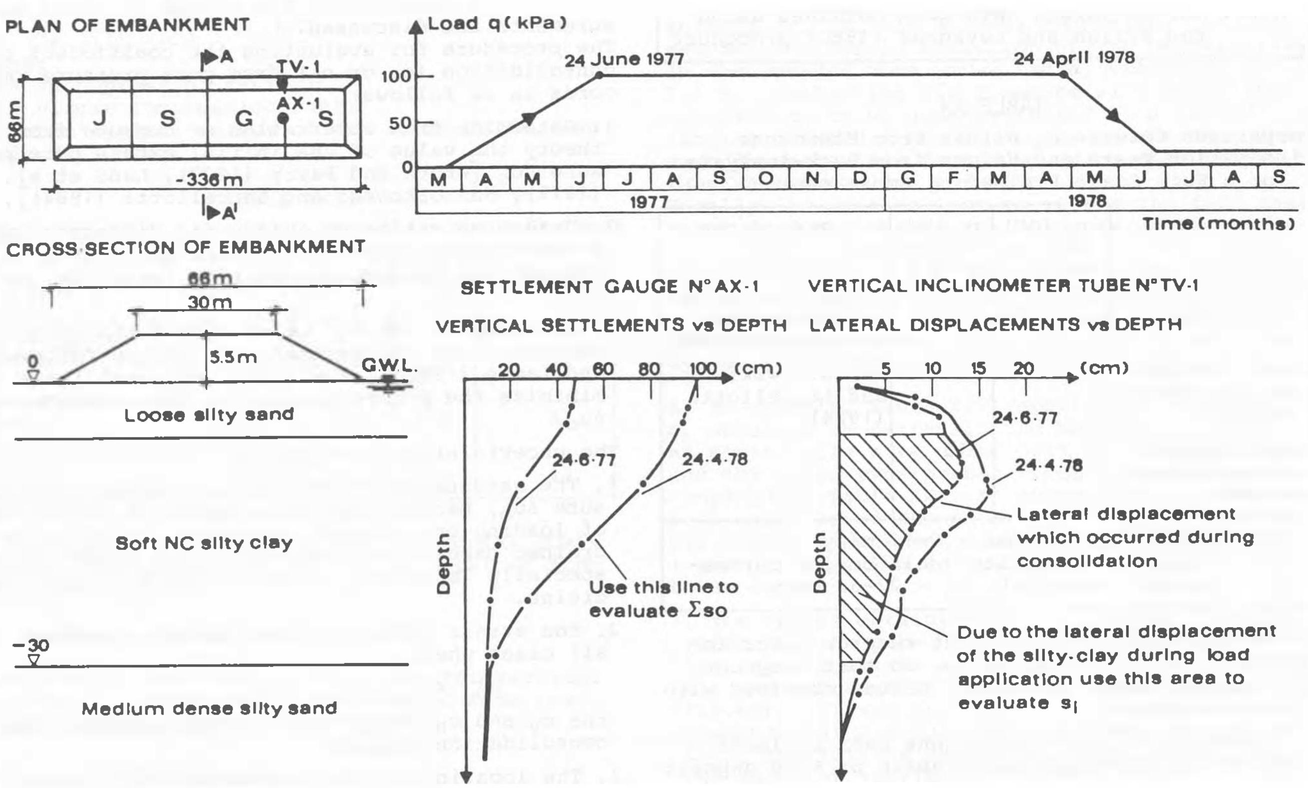



Fig.77: Example of Data Needed for Evaluating $c_h$ from Settlement Measurements [Jamiolkowski and Lancellotta (1984)].

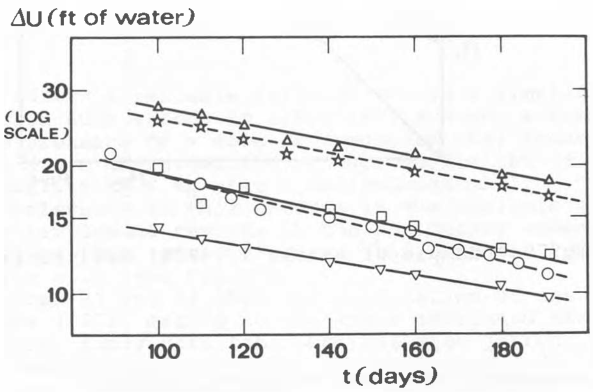
degree of consolidation at the end of the loading ramp which appears too low if it is compared to that deduced from observed consolidation settlements (see experimental data shown later in this section).

3. The evaluation of $\Delta u_o$ by means of the modified Cam Clay approach leads to $\Delta u_{max}/\Delta u_o$ ratios which are in agreement with the values of the consolidation settlement.

The ratios of $\Delta u_o/\Delta\sigma_v$ which are consistently higher than 1 may be attributed, at least partially, to the occurrence of contained plastic flow and strain softening phenomena.
The difficulties involved in evaluating $\Delta u_o$ as well as the uncertainties due to the location of piezometers relative to vertical drains may be bypassed by applying Asaoka's (1978) procedure to pore pressure records. This has been done by Orleach (1983) for fourteen piezometers installed in a 65 ft thick layer of plastic deltaic clay located in Mobile, Alabama, during a preloading project which involved which drains. From Barron's (1948) theory for pure radial drainage and the equal strain condition, it can be shown that the excess pore pressure $\Delta u$ at any time t may be expressed as:

$$\ell n\ (\Delta u) = \alpha_o - \alpha_1 t$$

where $\alpha_o$ and $\alpha_1$ are constants which can be determined by linear regression between $\ell n(n)$ and t (see Fig. 78).




| Piez. No. | El.(ft) | Symbol |
|---|---|---|
| SP-2 | -9.0 | △ |
| SP-3 | -9.9 | ☆ |
| SP-5 | -31.6 | ▽ |
| SP-6 | -30.7 | □ |
| P-2 | -9.9 | ○ |

Fig.78: Asaoka's (1978) Procedure Applied to Pore Pressure Records [Orleach (1983)].

The constant $\alpha_o$ is related to the product of the initial excess pore pressure $\Delta u_o$ and the location of the piezometer. The coefficient of consolidation $c_h$ is obtained from the slope $\alpha_1$ using the following relation:

$$c_h = \frac{d_e^2 \cdot F(n)}{8} \alpha_1$$

where:

$d_e$ = diameter of the drained soil cylinder
$F(n)$ = $n^2/(n^2-1) \cdot \ell n(n) - 0.75$
$n$ = $d_e/d_w$
$d_w$ = equivalent drain diameter

An inspection of the above formula shows that the computed $c_h$ value is independent of the initial excess pore pressure and the location of the piezometer tip.
When applied to the case of vertical drainage only, a similar equation can be derived to obtain the $c_v$ coefficient:

$$c_v = \frac{4\ H_d^2}{\pi^2} \alpha_1$$

where:
$H_d$ = drainage distance
$\alpha_1 = \ell n\ (u_1/u_2)/(t_2-t_1)$

But, because this equation is based on the Terzaghi approximate solution [see Orleach (1983)]. Its use must be restricted to time factors $T_v$ higher than 0.1.

A comparison between the $c_h$ values obtained using the above procedure with those from more conventional analysis is reported in Table XXII for the project in Mobile, Alabama [Noiray (1982), Orleach (1983)]. The $c_h$ values obtained from settlement records following the procedure described below are also listed.

TABLE XXII

$c_h$ Values Obtained from a Back-Analysis of Pore Pressure and Settlement Data [Noiray (1982; Orleach (1983)]

| Approach | From | $c_h$ ($m^2/y$) |
|---|---|---|
| Total elapsed time | Pore pressure data | 1.3 ± 0.6 |
| Incremental time | Pore pressure data | 1.3 ± 0.3 |
| Asaoka (1978) | Pore pressure data | 1.4 ± 0.3 |
| Total elapsed time | Settlement data | 4.7 ± 1.0 |
| Incremental time | Settlement data | 3.7 ± 0.3 |
| Asaoka (1978) | Settlement data | 4.4 ± 0.5 |
| Note: Laboratory $c_v$ = 0.7 ± 0.3 | | |

When evaluating the $c_h$ or $c_v$ from settlement data, reference is generally made to the ratio $\rho_c(t)$ over $\rho_{cf}$.

where:
$\rho_c(t)$ = measured consolidation settlement at time t
$\rho_{cf}$ = computed final consolidation settlement

The other steps are analogous to those described above for evaluating $c_h$ from pore pressure records (i.e., total and incremental time methods). But again the settlement approach also suffers from many uncertainties. The most relevant are the following:

1. A reliable estimate of $\rho_{cf}$ of the consolidating clay layer is difficult. This difficulty may be partially overcome by extrapolating the observed field measurements to obtain a probable value of $\rho_{cf}$.
2. For obtaining a reliable value of $\rho_c(t)$ for the clay stratum, all components of the surface settlement due to any pervious layers must be subtracted (see Fig.77).
3. In determining the part of the measured settlement which is really due only to consolidation, the immediate surface settlement $s_i$ must also be subtracted (see Fig.77). To do this accurately, it is suggested to measure the magnitude of horizontal displacements of the loaded area by means of vertical inclinometers, assuming any lateral displacement measured beneath the periphery of the loaded area must result in a settlement of the soil surface because the lateral deformation of the clay occurs at a constant-volume.

Also in this case, the difficulties related to the prediction of the initial settlement and of the final consolidation settlement can be overcome by using the procedure suggested by Asaoka (1978). When applied to consolidation problems in presence of vertical drains, this method is based on the Barron's (1948) solution for pure radial drainage. The relevant steps can be summarized as follows (see Fig.79):

1. From the time-settlement curve select a series of settlement values $\rho_1$, $\rho_2$ .....$\rho_n$, such that $\rho_n$ is the settlement at time $t_n$ and that the time interval $\Delta t = (t_n - t_{n-1})$ is constant [Fig.79(a)].
2. From this series, plot the points $(\rho_{n-1}, \rho_n)$.
3. All points lie on a straight line, such that:

$$\rho_n = \rho_o + \beta \rho_{n-1}$$

where $\rho_o$ and $\beta$ are two constants which depend on the selected time interval $\Delta t$.
4. The constant $\beta$ represents the slope of the constructed straight line, and it can be related to the coefficient of consolidation by:

$$c_h = - \frac{d_e^2 \, F(n)}{8} \cdot \frac{\ln \beta}{\Delta t}$$

The theory shows that the value of $c_h$ is independent of the time interval and the time of origin chosen.

The same approach can be used for consolidation with vertical drainage only. The relation between the slope $\beta$ and $c_v$ is [Magnan and Deroy (1980)]:

$$c_v = - \frac{4 \, H_d^2}{\pi^2} \cdot \frac{\ln \beta}{\Delta t}$$

This relationship is only valid for $T_v > 0.1$.

It must be stressed that Asaoka's (1978) procedure generates a straight line in Fig.79(b) only if the soil behaviour fulfills the Terzaghi theory assumptions. The actual plot (see Fig.80) may deviate from linear leading to an initial upward curvature, when:

- the condition of 1-D pore water flow are not fullfilled
- $c_h$ or $c_v$ changes during the consolidation process;

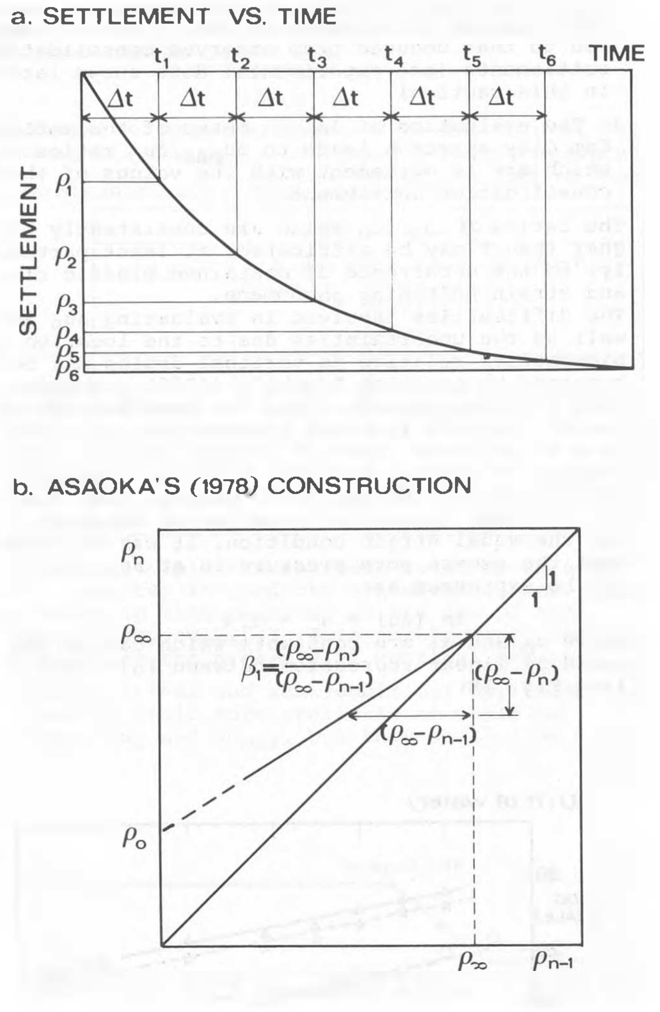


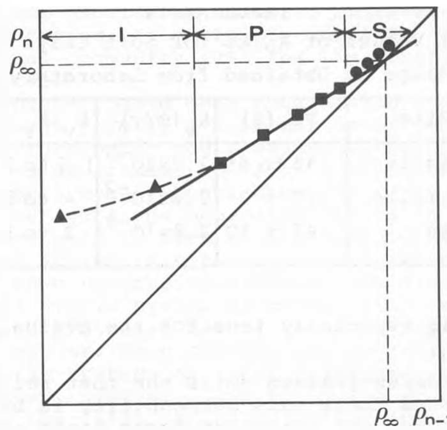

Fig.79: Example of Asaoka's (1978) Construction.

- secondary compression starts to occur.

This approach was applied by Orleach (1983) to the experimental data from the wick drain project in Mobile, Alabama. The results were reported in Table XXII, and the comparison with those from pore pressure data confirm the quite general trend found in the literature:

1. The field $c_h$ is considerably higher than $c_h$ resulting from conventional laboratory tests.
2. The $c_h$ back-figured from pore pressure records is lower than those computed from settlement records [Ladd et al. (1972), Adachi and Todo (1979), Choa et al. (1981), Holtz and Broms (1972)]. This can be partly explained by theory (i.e. compare coupled vs. uncoupled consolidation theory), but also by field instrumentation performance [Orleach (1983)].

Finally, when dealing with data from both excess pore pressure or settlement records from cases where vertical drains have been installed, there is a need of evaluating the smear effect in order




| ZONE | SYMBOL | STATE |
|---|---|---|
| I | ▲ | INITIAL STAGE OF CONSOLIDATION ($c_h$ OR $c_v$ DECREASING) |
| P | ■ | PRIMARY CONSOLIDATION WITH CONSTANT $c_h$ OR $c_v$ |
| S | ● | SECONDARY COMPRESSION |

Fig.80: Possible Deviations from Asaoka's Construction [Orleach (1983)].

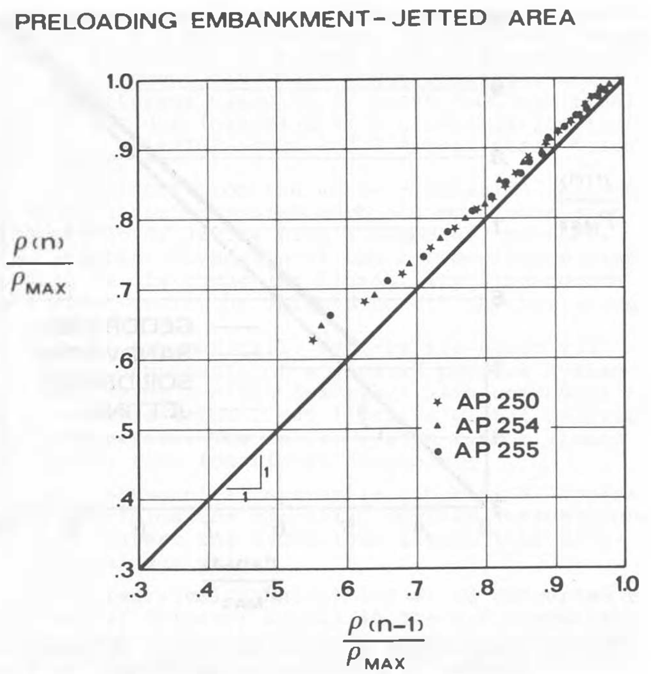



Fig.81: Asaoka's (1978) Procedure for Determining the $c_h$ Value on Porto Tolle Silty Clay.

to assess a reliable value of $c_h$. In a simplified design such effect is taken into account assuming the presence of a zone of remoulded clay around the drain characterized by the coefficient of permeability $k_r < k_h$ of the undisturbed soil.
Of relevance in this context is the analysis of the settlement records of the preloading embankment on four types of drains built at the Porto Tolle site (see Fig.77).
Figures 81 and 82 show the application of the Asaoka (1978) method to the above mentioned case record. (Only data from sections with jetted sand drains and Geodrains are here reported).
It must be stressed that in order to minimize the errors involved in the determination of the slope β, the data have been normalized with respect to the largest measured settlement $\rho_{max}$, so that regression lines can be obtained from a very large number of experimental data. The regression lines as obtained for the different drain areas are compared in Fig.83, and the deduced $c_h$ values are given in Fig.84. This figure shows that by making an obvious assumption that the same $c_h$ holds for all four examined areas, one gets $k_h/k_r$ values ranging between 1.5 and 2.0 which are well below those from laboratory tests on completely remoulded specimens; see Table XXIII.

3.4.7. Summary and Conclusions

1. The macro and microfabric of natural soils gives them a significant anisotropy with respect to their flow and consolidation properties which makes the assessment of the k and c uncertain from both laboratory and local in situ tests.

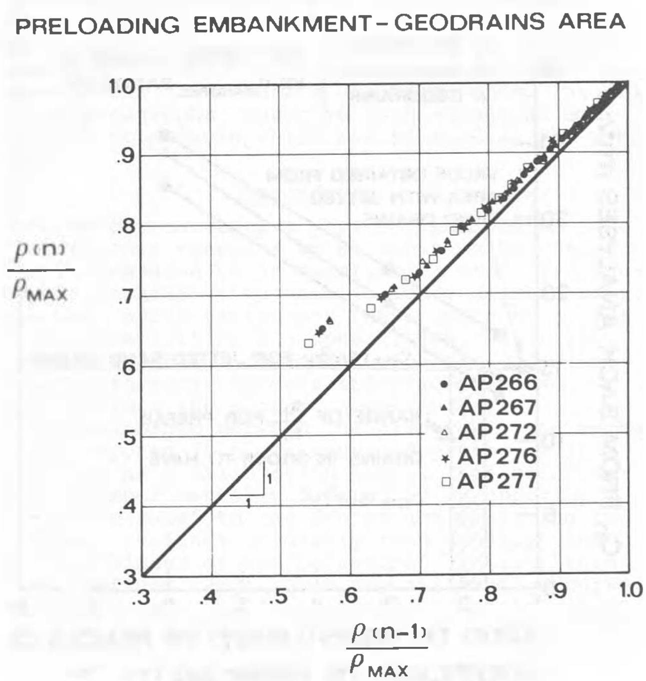



Fig.82: Asaoka's (1978) Procedure for Determining the $c_h$ Value on Porto Tolle Silty Clay.

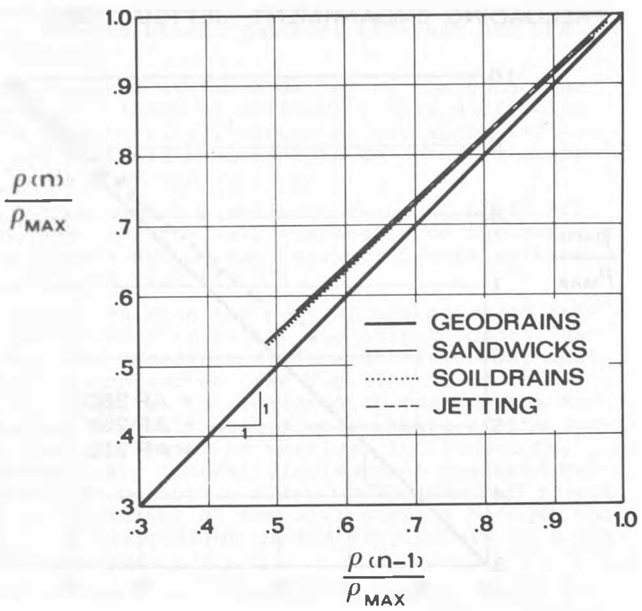



Fig.83: Comparison Between the Rates of Consolidation of a Preloaded Embankment on Four Types of Drains (Porto Tolle Silty Clay).

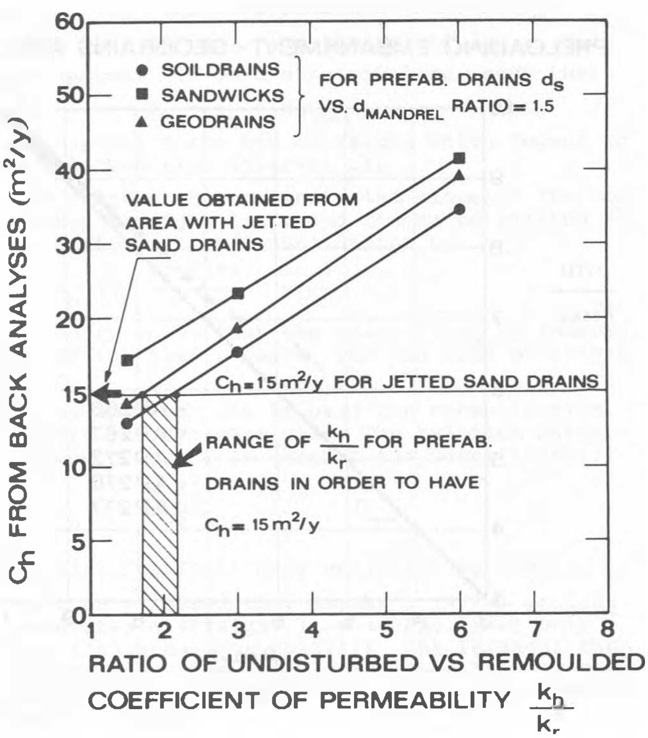



Fig.84: Values of $c_h$ Deduced from Regression Lines of Fig.83.

TABLE XXIII

Typical Values of $k_v/k_r$ for Soft Clays in the NC Range as Obtained from Laboratory Tests

| Site | PI (%) | $k_v$(m/y) | $k_v/k_r$ | $m_v/m_r$ |
|---|---|---|---|---|
| Panigaglia | 45 to 65 | $2.2\times10^{-3}$ | 1.5 to 3 | 1.45 |
| Porto Tolle | 31 ± 2 | $2.2\times10^{-2}$ | 4 to 8 | 1.21 |
| Trieste | 47 ± 10 | $2.8\times10^{-3}$ | 2 to 3 | 1.33 |

This is especially true for the evaluation of $c_h$ and $k_h$.

2. In coarse-grained soils the most reliable way to assess their mass permeability is by properly programmed and conducted large scale pumping tests.

3. In these soils the borehole outflow and inflow permeability tests are also used but they yield coefficients of permeability which are at best only representative to an order of magnitude of the $k_h$ or $k_v$ in situ.

4. When assessing k and c in situ in cohesive deposits, it is essential to pay proper attention to the range of the effective stress state within which each specific test method is operating. All outflow tests performed with piezometers lead to k and c values relevant to cases where $\sigma'_v < \sigma'_{vo}$, while inflow tests yield k and c values which are appropriate to the condition of $\sigma'_v > \sigma'_{vo}$. None of the present in situ techniques allows the k and c values to be determined in the range of $\sigma'_v$ well beyond $\sigma'_{vo}$.

5. The recently introduced self-boring devices minimize to a great extent the disturbance of the soil in which k and c are to be assessed. At present only outflow tests can be done with the self-boring permeameter. There is some uncertainty about $c_h$ obtained from "holding" tests with the Camkometer because of nonmonotonic changes of the effective stress around the expanded SBP probe.

6. The interpretation of piezocone dissipation records is still subject to theoretical uncertainties. Among them the one related as above to nonmonotonic changes of the effective stress state around the probe during $\Delta u$ decay. This uncertainty seems to be particularly relevant from a practical point of view.

7. At least in principle, the most reliable information concerning k and c may be deduced from appropriate analyses of the strains and excess pore pressure changes vs. time measured under full scale structures or prototypes. This approach however requires some preliminary assumptions of e.g. $s_{cf}$, $\Delta u_{max}$, and $s_i$ to be made which affect the reliability of the deduced k and c values.

## 4. DATA ACQUISITION SYSTEMS

### 4.1. INTRODUCTION

The use of microcomputer based data acquisition systems is becoming much more common in geotechnical laboratories and in situ testing programs. In a recent survey conducted by ASTM Committee D-18 of 65 ASTM member laboratories (50% of which were government or university related), 37 were using some form of a data acquisition system. While this may not provide an accurate cross-section of all laboratories, it certainly confirms that the professional community realizes the potential of such devices.

A microcomputer based data acquisition system in its simpliest form is a combination of two devices; a microcomputer and a measuring device. The computer is programmed to watch a clock until it is time for a measurement. When the time arrives, the computer asks the measuring device to take a reading and send it back to the computer where it is then stored in a retrievable location. The computer then returns to watching the clock.

While this simple system is essentially complete, it makes very limited use of the full capability of the computer. Many options and devices can be added to the system, as later discussed. The example does, however, completely and accurately describe the first and primary function of data acquisition: to obtain and store measurements at specified times or events. If a system provides one hundred optional features but cannot 100% fulfill this requirement, then it is of little value for data acquisition.

Microcomputer based data acquisition systems can be used for three categories of testing:

1. For tests which are performed with feedback control in a relatively slow environment (e.g. the directional shear cell, stress path triaxial cell, controlled hydraulic gradient consolidometer). In such systems the data acquisition unit provides a report on the present sample state and sends control signals to perform test functions.

2. For testing with multiple output devices such as triaxial cells, piezocones or constant rate of strain consolidation. For these devices collecting data with analog strip chart records, x-y plotters or voltmeters is not practical.

3. For testing with combinations of devices in a general laboratory environment, where the system can monitor several experiments simultaneously.

Categories 2 and 3 are of most general interest since they can both be serviced by the same system if properly configured. Category 1 devices generally require a dedicated system which is specially programmed for the specific test.

Table XXIV summarizes the measurement needs of several field and laboratory devices. The measurement column sums up the minimum number of measuring devices needed for each test, plus some options. For example, a triaxial test with pore pressure requires measurement of pore pressure, cell pressure, vertical load, and vertical displacement (four devices). In addition to taking readings of these devices, the system must record the input voltage and time. Therefore, each scan collects six measurements from the "A/D converters" (described shortly). The time interval between scans also depends on the test type, rate of testing and the purpose of the test. Consolidation readings are taken on a variable time scale which follows a square root or logarithm of time scale. In contrast, triaxial readings are collected at constant time intervals which are increased several times during the test. Finally, an estimate of the total number of scans required for the test is given.

Specifications for a particular system must consider all of the above factors. For example, consider a data acquisition system to monitor three consolidation, one direct simple shear and two triaxial tests. Such a system must have a minimum scan interval of 4 sec (controlled by the consolidation test), a total of 19 signal input channels (one per measurement plus four power readings) and a minimum storage capacity of about 8000 numerical values per day (assuming 24 h consolidation increments, 1 triaxial shear and 3 direct shear tests as typical).

It is apparent from the above discussion that the data acquisition system combines much more than one measuring device with a computer. In fact, the greatest advantage of the microcomputer based system is its component flexibility. The components can easily be changed to fit the individual needs.

Figure 85 schematically depicts the basic and optional components of a general purpose system. The following section describes each component in considerable detail. But first, a global overview is appropriate. The entire system can be viewed as having four functional components.

1. The geotechnical system (e.g. triaxial apparatus) contains the measuring devices (transducers) which convert the mechanical action into an electrical equivalent.

2. The electrical (analog) signal is converted to a digital (binary) signal in the A/D converter. This unit also contains the computer controlled switching mechanism (scanner) which connects the proper wires to the converter.

3. The computer orchestrates the components and performs the administrative and computational tasks.

4. The peripheral components include anything from a modem (telephone connection to a second computer) to a graphics plotter. These represent the custom aspects of the system's flexibility.

## 4.2. HARDWARE

Microcomputer based data acquisition systems can range from a single self contained unit to a complex network of computers and components. The following paragraphs describe both essential and optional components which can be used in the system.

### Transducer

A transducer converts a physical phenomenon into an electronic response which can then be conveniently measured by an electronic meter. Transducers are available to measure load, displacement, pressure, acceleration and temperature in a wide range of capacities and sensitivies. Generally, the sensitivity is dependent upon the capacity. Each manufacturer targets specific needs and hence tailors the product accordingly.

The relationship between the electronic and mechanical response (e.g. voltage and pressure) is expressed as a calibration curve. This relationship is experimentally determined by applying a known "pressure" to the device and measuring the resultant "voltage". Ideally (and usually) the curve is linear or can be assumed linear within a well defined range. Once the calibration curve is established, the "pressure" (P) can be calculated from the measured "voltage" (V) using:

$$P = CF \cdot (V - Z)$$

where the calibration factor is the slope of the line and the zero (Z) is the output of the transducer at zero pressure. CF is usually normalized to the transducer excitation voltage to eliminate the adverse effect of small power fluctua-

TABLE XXIV

Data Acquisition Requirements for Typical Laboratory and Field Devices

| Device Type | Measurements Per Scan * | Scans Per Test | Scanning Interval |
|---|---|---|---|
| Permeability | | | |
| Falling head | 1 | 20-30 | 10 sec - 5 min depending on k and increasing with time (t) |
| Constant head | 3 (min.) | 100-1000 | 10 sec - 5 min depending on k |
| Consolidation | | | |
| Incremental | 1 (min.) | 150/24 hr incr. | 0.4 sec - 10 min (increase with time) |
| Constant rate of strain | 4 | 500 | 1 min - 10 min depending on permeability |
| Controlled gradient | 4 | 500 | |
| Incremental with lateral stress | 2 | 150/24 hr incr. | same as incremental |
| Shear | | | |
| Direct shear | 3 | 100-200 | 15 sec - 10 min } depending on $\dot{\varepsilon}$ and increasing with t |
| Direct simple shear | 4 | 100-200 | 15 sec - 10 min |
| Torsional shear | 4 | 100-600 | 15 sec - 10 min |
| Triaxial | | | |
| unconfined | 2 | 50-100 | 15 sec - 1 min depending on $\dot{\varepsilon}$ and increasing with t |
| confined | +1 | 100 to 1000 | 1 min - 10 min depending on $\dot{\varepsilon}$ and increasing with t |
| pore pressure | +1 | | |
| volume change | +1 | | |
| lateral strain | +1 | | |
| True triaxial | 6 (min.) | 100-1000 | 1 min - 10 min depending on $\dot{\varepsilon}$ and increasing with t |
| pore pressure | +1 | | |
| volume change | +1 | | |
| Plane strain | 3 | 100-1000 | 1 min - 10 min depending on $\dot{\varepsilon}$ and increasing with t |
| pore pressure | +1 | | |
| volume change | +1 | | |
| Directional shear cell | 12 (min.) | 100-1000 | 1 min - 10 min depending on $\dot{\varepsilon}$ and increasing with t |
| pore pressure | +1 | | |
| intermediate stress | +1 | | |
| Hollow cylinder | 6 (min.) | 100-1000 | 1 min - 10 min depending on $\dot{\varepsilon}$ and increasing with t |
| pore pressure | +1 | | |
| volume change | +1 | | |
| Cone penetration | 3 | 0.5 to 5/cm | 0.1 sec - 1 sec depending on scan rate |
| pore pressure | +1 | | |
| temperature | +1 | | changing to variable time for dissipation |
| lateral stress | +1 | | |
| Pressuremeter | | | 10 sec - 30 sec depending on expansion rate |
| Camkometer | 5 | 100-200 | |
| PAFSOR | 2 | | |
| Field vane | 2 | 50-100 | 10 sec - 30 sec depending on $\dot{\varepsilon}$ |
| Inclinometer | 2 | 50-100 | manual trigger |

* Computer must also read input voltage and time for each scan

tions. The output of the transducer at zero "pressure" is usually not numerically zero and will depend on the manufacturing process.
The precision of a transducer is evaluated by a number of measures (shown graphically in Fig.86) calculated from the calibration data, such as the coefficient of determination ($r^2 \geq 0.9999$), repeatability, nonlinearity, hysteresis and temperature stability. The combination of these measures gives the overall precision of the transducer which must be evaluated separately for each application. For example, the precision of an incremental value is much greater than that of a long term cyclic total value.

## Power Supply

Power supplies are used to excite transducers with either constant current or constant voltage. Integrated data acquisition systems usually contain the power supply used with the recommended transducers. Several transducers can easily operate on a 0.5 watt source. The power supply must be stable (i.e. give a constant output and have little noise). Any power irregularity will be amplified by the transducer and appear in the final output signal.

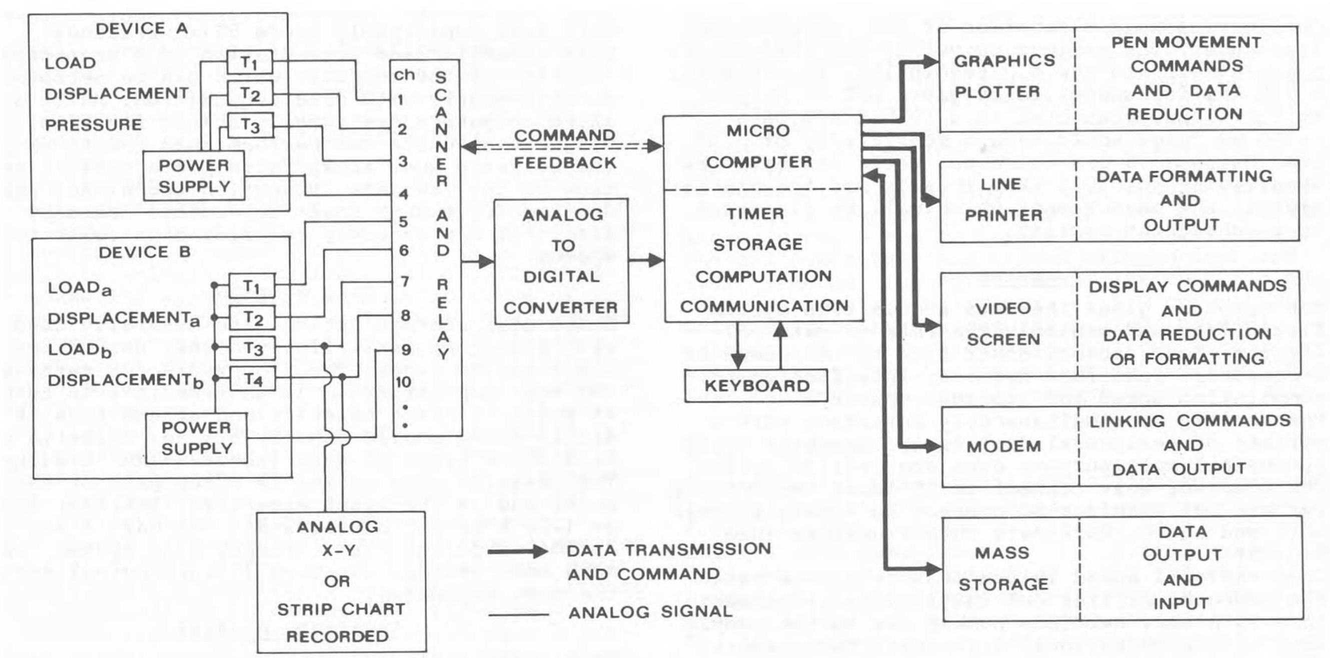



Fig.85: Schematic Diagram of the Basic and Optional Components of a Data Acquisition System.

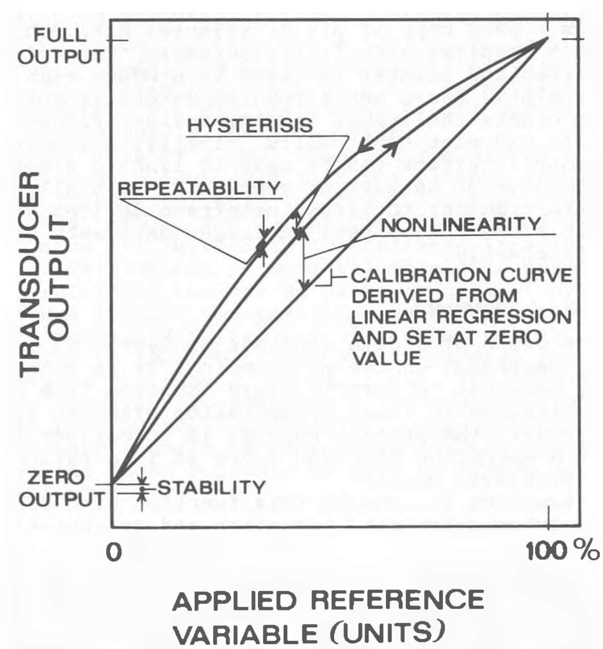



Fig.86: Definition of the Various Measures of Transducer Precision.

## Scanner

The scanner is a relay system which switches the feeds (outputs) from transducers to the analog to digital (A/D) converter. The scanner receives command inputs from the computer selecting the transducer to be activated. The scanner makes the switch and alerts the computer that the connection is complete. High quality connections are essential to low noise, stable readings. In noisy environments (machinery, long electrical leads, radio transmission, etc.) signal stability can be greatly improved with a scanner that switches the individual ground wire as well as the high and low signal leads.

## Analog to Digital Converter

This unit converts the transducer output into a form which is recognizable by the computer. Once converted, the signal is no longer subject to degradation by transmission losses or noise (see 4.5.). Therefore, it is advantageous to make the conversion as close to the transducers as practical and then transmit the digital signal.
The A/D converter establishes the sensitivity of the system. It is a binary encoder which divides the voltage into a number of components. The precision of conversion depends on both the number of divisions and the range (size) of the scale. The minimum acceptable conversion is 12 bit resolution which provides $2^{12}$ increments (1 part in 4096). More desirable devices provide 16 bit (1/65 536) or even 18 bit (1/262 144). The second variable, the range, makes use of an amplifier with predetermined gains. Most devices have a maximum amplification of 100 times using 4 or 8 gain options allowing full scale ranges between

100 mv and 10 v. The sensitivity of a particular configuration is a function of the range of the transducer, the maximum output of the transducer, the A/D gain and the A/D resolution. For example, a 700 kPa transducer which gives 100 mv output at full load, connected to a 12 bit A/D on a ± 100 mv range would have a sensitivity of 0.33 kPA. Using a 16 bit converter would increase the sensitivity to 0.021 kPa. This is not the precision of the measurement which will be discussed in a subsequent section.

### Computer

The computer gives the data acquisition system flexibility and controls the rate of data collection. Four factors contribute to the power of a computer: interface options, interface speed, computation speed and storage capacity.
The ability to simultaneously interface with a variety of peripheral devices is essential to an economical multipurpose data acquisition system. The computer must connect to at least the A/D converter, but should also connect to a mass storage unit and one or more data output devices (see Fig.85).
Computational speed is one measure of how fast the computer carries out instructions. However, this is a very nebulous number due to the complexity of the operational structure. For example, the Intel Z80 chip uses a clock cycle of 250 nsec to pace the operation rate. The minimum instruction execution time is from 1 to 5.75 μsec or 4 to 23 clock cycles. This rather wide range reflects the time requirements of the different operations and illustrates the difficulty in determining a unique computation time. This range (174 000 to 1 000 000 operations/sec) will not be the controlling factor for data acquisition applications.
The interface speed creates a more practical limitation. The computer must collect the data from the A/D converter and then store it. The actual time required to collect data is dependent on the sequence of operations (software). The fastest possible method is to collect a reading and hold it in virtual memory (in the computer). Of course, this is lost when the computer is turned off. Typical reading rates for this scenario range from 300 to 2 000 readings/sec, a small fraction of the computational speed. A second (and slowest) linear scenario is to take a reading and store it permanently on an external device (disk, tape, etc.) before returning for the next reading. In this case, the rate is essentially dependent on the storage unit access time which is from a fraction of a second to several seconds. Many intermediate rates can be achieved by collecting groups of data but will result in short shutdown periods during data transfer.
A third very powerful variation involves the use of multitasking which is the ability to perform simultaneous operations. In this case one section of the computer collects data into virtual memory while a second section transfers numbers to storage, allowing collection rates up to 20 or 30 readings/sec without interruption or data backup in memory.
Computer storage capacity is seldom a restriction in normal read-store applications. However, when data are to be temporarily held in memory (fast data collection) prior to permanent storage, space allocation may become critical. Computer storage memory can be expanded from 64 to 516 k bytes or more with extension modules (storage of one numerical value requires 8 bytes). Allowing space for program lines and variables, a 516 k unit will confortably store 55 000 readings.
Data reduction and presentation is a secondary function of the computer which can be performed simultaneously with data acquisition. Multitasking computers are ideally suited for this. Single task units can perform both functions but the software must always give preference to storage of the raw data. Reduction techniques seldom require enough space to dictate computer size, but may severely restrict data collection speeds.

### Mass Storage

Three mass storage options are generally used with microcomputers: floppy discs, hard discs and cassette tapes. The floppy disc is certainly the most popular, and it is intermediate in terms of price, storage capacity and access time. A single-sided double density 5.5 in. diskette will hold 300 k bytes of data (about 35 000 readings). The cassette tape option is often part of the computer and is the least expensive, smallest storage (200 k bytes) and slowest. The hard disc option is faster than a floppy disc system, has much more storage capacity (5 000 k bytes) and is the most expensive.

### Ancillary Devices

Four additional devices are useful for data manipulation and display: a line printer, a graphics screen, graphics printer, and communication interface. All these devices require software to operate and hence must share time with the data collection task. For computers which do not multitask, assuring timely data collection is a formidable task.
The line printer is almost essential as it provides a hard copy of all or selected data. This unit requires very little software. The graphics screen and printer are used to produce high quality plots. These units require extensive software to create the frame, labels, scales, reduce the data and plot the results. Finally, a communications interface can be used to link to a second computer or network of computers. This allows data transfer to larger mainframe devices which can store and operate on larger data sets very efficiently.

## 4.3. SOFTWARE

The value of a data acquisition system is largely dependent on the programming. It is conceptually possible to perform every function from data collection to final presentation with one system. However, the primary purpose is to collect data at a specified time and store it in a rational retrievable manner.
Attempting to combine this function with elaborate interactive data reduction and presentation schemes is likely to create problems. For heavily used systems it is best to separate the real time data collection from the virtual time computation and presentation functions. This is easily accomplished with compatible mass storage units or direct interfaces.
While many people choose to write customized programs, several packages are commercially available. These packaged programs range in application from simple data collection and storage to geotechnically oriented test collection and presentation, to specialized graphics and statistical reduction.

## 4.4. SIGNAL CONDITIONING

Signal conditioning is a modification of the transducer output such that a second calibration curve is required to calculate the response. Signal conditioning is adapted to either improve measurement precision (amplification) or make the calibration more convenient.
Amplification should only be considered when absolutely necessary because it adds another variable, the amplification factor, to the conversion relationship. Measurement errors must be evaluated prior to selecting the amount of amplification.
A measuring system is evaluated in terms of accuracy and precision. The system can generally be presumed accurate as long as it is calibrated to an accurate standard. Therefore, the calibration curve makes the transducer accurate (i.e. the average reading equals the true value). In an accurate system, precision expresses the scatter of the measurements about the real value. This is quantified as nonlinearity, stability, repeatability and hysteresis for transducers (Fig.86) and sensitivity and noise for A/D converters.
The amount of beneficial amplification (improved precision) will depend on the transducer, power supply and A/D converter. For example, a transducer giving 35 mv output at full pressure with a combined precision of ± 0.01% would have a reading precision of ± 0.0035 mv. This unit is connected to a 12 bit A/D converter having ± 100 mv range and ± 1 bit of noise and hence a precision of ± 0.097 mv. For this system, the signal can be amplified by as much as 30 times before the transducer precision becomes critical.
A more subtle form of amplification occurs within some A/D converters to obtain as precise a reading as possible. The actual conversion is performed on a single scale (± 10 v) but the converter can amplify the signal by predetermined magnitudes to almost completely fill this scale. The system keeps track of the gain value and then alters the digital signal by the gain factor so the output is the true value.
Another type of signal conditioning alters the transducer output such that the voltage reading more closely reflects engineering units (pressure, strains, etc.). The calibration factor and/or zero value are physically incorporated into the system. This method is most valuable with analog x-y recorders but has no place in digital systems because it adds two variables which cannot be backfigured. More importantly, these variables do nothing to improve the signal. Recording the true voltages provides a traceable path and information necessary to evaluate a transducer's long term performance. In addition, computers can convert pure signals to engineering units so quickly that the added hardware is more of a problem than a solution.

## 4.5. NOISE

Unlike voltmeters which average readings over relatively long periods of time (e.g., 0.5 sec), A/D converters capture essentially instantaneous measures of the signal. Therefore, high frequency irregularities (noise) in the signal are much more detrimental to these latter devices, resulting in a decrease in precision. Noise can be due to a variety of causes such as the AC power source (60 Hz hum), loose connections, poor grounding, ground loops, or a faulty device. Finding and correcting the problem can range from very difficult to virtually impossible. Provided that all the devices in the system are functioning adequately, there are three operational improvements which reduce noise.
Improper grounding is the most common cause of noise. Each pair of signal wires should be wrapped with a shield. This protection should pass individually through junction boxes (not one common ground for the box) and continue on with the wire.
In general, the grounding network should branch out like a tree with one common sink thus avoiding group loops. Noise is further reduced by having the A/D converter read ground between each and every reading. This provides a sink to drain any residual power. Finally, use of A/D converters which switch three wires (high, low, ground), rather than the more conventional two wire relays will greatly improve the situation. However, these last two measures greatly reduce scanning rates.

## 4.6. GENERAL EVALUATION

Microcomputer based data acquisition systems offer several advantages over analog recorders or manual documentation. Besides collecting more complete and reliable measurements, the systems can collect information at variable time rates yet maintain essentially instantaneous measurements. These systems can collect data at rates appropriate for the test purpose, i.e. fast for short test or more slowly for long duration experiments. As an example, Fig.87 illustrates the use of a data acquisition system to check the pore pressure response (quality of de-airing) of a fine ceramic stone which has been saturated with water. The stone is epoxied in the base plate of the device which has been enclosed in a pressure vessel for this experiment. A witness transducer is used to monitor the water pressure in the chamber. Two pressure pulses are generated in the chamber by displacing a piston. The data acquisition system reads these two transducers at a rate of 300 scans per second, storing the data directly in virtual memory and then transferring the measurements to disc.
The figure shows the actual readings (millivolts) versus time. These transducers have different calibration factors and input voltages as indicated in the figure. The adequacy of de-airing is judged by:

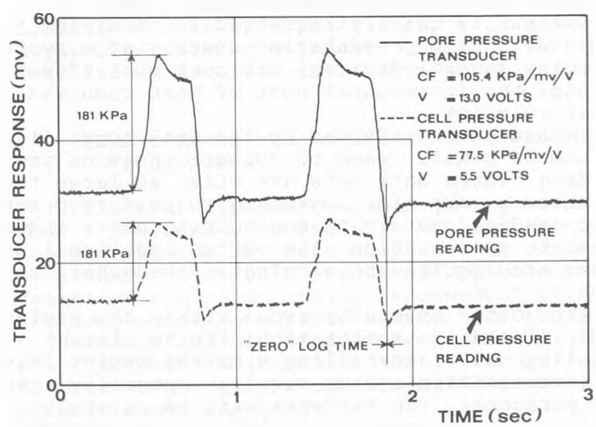



Fig.87: Pressure Transducer Response Versus Time as Measured by a Data Acquisition System.

1. the response values (Δ pore pressure / Δ cell pressure) and

2. the lag time (time between witness attaining a state and pore pressure attaining same state). In this case, the response value is 100% and the lag time is less than one reading/scan (< 1/300 sec).

This same system can be very quickly reconfigured to monitor a cyclic direct simple shear test or taken to the field for piezocone penetration.
In addition, data acquisition systems operate overnight or for extended periods without causing serious planning or personnel problems.The data are always consistently documented. The collected measurements more reliably reflect the transducer state because the data acquisition system does not depend on mechanical motion.
Independent of the rate of data collection, the individual numbers reflect instantaneous measures of the transducer (Fig.87). This eliminates uncertainty created by relatively slow responding recorders.(Typical strip chart recorders have a 0.5 sec response time which would essentially miss the two pulses in this figure.) The combination of hardware and software flexibility create the most important attribute of the system. Components can be quickly changed for special tests or the computer reprogrammed to convert from one device to another. Large systems can service many tests and also provide many user options.
Digital data collection and storage allow optimum use of the computers' computational power. Data can be analyzed as it is collected to facilitate quick review. Conversion of signals to engineering units eliminates the need for conditioning or digitization. The data are immediately stored in a form which can be recalled for parametric studies or future analysis.
While data acquisition systems generally improve our capability, there are several drawbacks which must be considered. The most troublesome concern is the presence of signal noise. The quick access time of the A/D converter allows it to measure the instantaneous signal including any irregularities. The reading precision is often a limitation for less expensive systems, which then requires additional signal amplification. In addition, the systems require temperature stability, a clean environment, low humidity and a stable power supply. The initial cost of small systems is much higher than for recorders and multimeters, but the investment is quickly recovered in recording time, computation and presentation costs. Large systems (reading several devices) are most cost effective because the incremental cost of more channels is relatively low.
The added power provided by the data acquisition system is usually used to collect enormous amounts of data. These data sets are often so large that it becomes very time consuming to perform parametric studies,convert to engineering units and condense to presentation size. Often additional programs are required to eliminate unnecessary numbers.
As with other specialty areas within the profession, (field instrumentation, finite element modeling, etc.) installing a microcomputer based data acquisition system requires specially trained personnel. The hardware must be carefully selected to be compatible with the measurement needs and then the software structured to efficiently control the hardware. Small technical details can render the entire system non-functional.

### 4.7. FUTURE DEVELOPMENTS

#### Hardware

There are three developing areas of major concern which should be stressed:

- Noise level rejection devices (such as high, low, ground relays and fast draining relays) need to be improved and standardized in all A/D converters. This will greatly improve precision and eliminate the need for amplification.
- Increased sensitivity using at least 16 bit A/D converters combined with auto ranging (i.e. self selecting gain to give highest sensitivity) will maximize precision and again eliminate the need for amplification.
- Faster relays, A/D converters and interface handshaking to allow data collection at rates comparable to computer computation rates. Since minicomputers will soon be desktop size, the data collection task will again be 100 to 1000 times slower than the computer.

#### Software

Whereas geotechnical engineers must depend on others for hardware development, the profession can make significant contributions to data acquisition software. Again, three challenges are cited:

- Data reduction techniques need to go beyond simple linear calculations using average values. The computations should incorporate calibration nonlinearities,variations in sample geometry and measurement precision. The results would then reflect the total uncertainty, as well as an average value. This will provide a valuable measure of test quality.
- Data collection schemes which intelligently collect meaningful data rather than simply recording everything.
  The data acquisition system should decide what is significant and disregard the rest. This will greatly reduce storage needs and facilitate computation and presentation.
- Data management schemes to efficiently compile test results. Alarming quantities of test data are lost due to improper documentation and storage. These data should be stored in a usable, logical retrievable form so past measurements can be used to answer future questions.

## 5. FINAL REMARKS

Despite the extremely broad spectrum of the topics covered by the title of the present Theme Lecture, the writers shared the opinion of the organizing Committee of the present conference "that it is not meant to be a State-of-the Art Report". Consequently it covers a limited number of selected topics.
These topics have been selected taking into account the current research needs within the field of Experimental Soil Mechanics, but also paying due consideration to the research presently performed at the universities to which the writers belong. Within that frame the most relevant points related to the covered topics may summarized as follows:

## LABORATORY TESTING

### 1. PRECONSOLIDATION PRESSURE, YIELDING AND NORMALIZATION

1.1. The preconsolidation pressure $\sigma'_p$ of a soil represents a yield stress that separates small strain "elastic" behaviour from large strains accompanied by plastic (irrecoverable) deformation during one-dimensional compression. It is the single most important parameter controlling basic soil behaviour during both drained and undrained loading conditions and therefore is particularly suitable for normalizing relevant strength and stiffness characteristics of cohesive deposits.

1.2. Practitioners should place more emphasis on careful development of the $\sigma'_p$ profile for projects involving sedimentary clays. This will usually entail:

(1)
Use of end-of-primary compression curves from oedometer tests performed on high quality samples (Note: continuous loading consolidation tests if run at too fast a rate overestimate $\sigma'_p$.)

(2)
Interpolation-extrapolation via correlations with in situ penetration or field vane tests; and

(3)
Consideration of the geologic history of the deposit.

1.3. Item (3) above is important to help ascertain the possible physical mechanisms responsible for development of the $\sigma'_p$ profile. For example, mechanical overconsolidation affects the shape of the yield surface, the value of $K_o$ and other soil properties in a manner different from natural cementation.

1.4. Field and Lab data indicate that the in situ undrained strength appropriate for stability analyses is closely related to $\sigma'_p$ for most low OCR clays of low to moderate plasticity, independent of the preconsolidation mechanism. The relationship is $c_u/\sigma'_p = 0.23 \pm 0.04$

### 2. SAMPLE DISTURBANCE (COHESIVE SOILS)

2.1. Radiography is ideally suited to detect obvious signs of sample disturbance and to help select the most representative and best quality soil for engineering tests.

2.2. Unconsolidated-undrained strength testing cannot yield meaningful stress-strain data and often gives unreliable and highly scattered $c_u$ values due to varying degrees of sample disturbance and uncontrolled errors resulting from the effects of strain rate and anisotropy. In situ stress-strain properties should therefore be obtained by either reconsolidating test specimens to the in situ stresses, the recompression technique, or by using the SHANSEP approach.

2.3. The recompression technique:

(1)
Is clearly superior for highly structured deposits (e.g. brittle, sensitive Canadian clays).

(2)
Is preferred whenever block quality samples are available and for testing weathered and highly overconsolidated deposits where SHANSEP is difficult to apply; and

(3)
Should always be accompanied by measurements of the in situ stress history.

2.4. The SHANSEP technique:

(1)
Is strictly applicable only to mechanically overconsolidated and truly normally consolidated deposits exhibiting normalized behaviour.

(2)
Is probably preferred for testing tube samples from deep deposits of low OCR ordinary clays.

(3)
Has the advantage of developing normalized stress-strain strength parameters that can be used on subsequent projects.

2.5. Systematic studies comparing results from recompression and SHANSEP testing on block samples subjected to varying degrees of disturbance are needed as a function of soil type and in situ OCR.

### 3. ANISOTROPY

3.1. Sufficient data now exist to indicate that inherent anisotropy has a significant effect on the yield stress and plastic stress-strain properties of most natural soil deposits. This aspect of soil behaviour cannot a priori be ignored in undrained stability analyses or in development of realistic soil models for finite element analyses.

3.2. Detailed measurements of inherent anisotropy using highly specialized test devices are needed on a variety of soils to generate a "data bank" for continued development of better soil models. This research should include comparative studies of the behaviour of identical soil samples in different shear devices.

3.3. Designers can obtain a practical estimate of the effects of anisotropy in cohesive soils by performing triaxial compression and extension and direct simple shear tests of $K_o$ reconsolidated samples.

3.4. Soil models used to analyze problems involving large stress reversals and cyclic loading conditions must incorporate appropriate laws governing evolving anisotropy. Unfortunately, very little data exist for this purpose.

### 4. TIME EFFECTS DURING ONE-DIMENSIONAL CONSOLIDATION

4.1. Extensive laboratory research by Mesri and Choi (1985) and some reliable field data support the concept for any soft clay of a unique end-of-primary void ratio versus effective consolidation stress relationship independent of the drainage height and hence of the time required for dissipation of excess pore pressures. This important conclusion means that "viscous" creep effects do not play a significant role during primary consolidation.

4.2. Mesri and his colleagues also show that the abnormal pore pressure behaviour observed in the field for some highly structured clay deposits can be readily explained by nonlinear compressibility rather than some sort of creep phenomenon.

4.3. The writers present data indicating that $K_o$ remains essentially constant during secondary compression over log time cycles of practical concern.

## IN SITU TESTING

### 1. SOIL PROFILING

1.1. The existing in situ techniques (CPT, DMT, CPTU) offer cost-effective means for reliable and detailed soil profiling and identification. Piezocone tests present especially great potential in this respect provided that a sufficiently rigid and very carefully de-aired system is used to measure the penetration pore pressure.

### 2. IN SITU HORIZONTAL STRESS AND STRESS HISTORY

2.1. Measurements of $\sigma_{ho}$ in situ are still very difficult and of uncertain reliability.

2.2. Among the examined devices the SBP presents the greatest potential in this respect even if futher research concerning the importance of various details of equipment performance and insertion procedure are warranted.

2.3. Present experience with SBP in clays leads to "reasonable" values of $\sigma_{ho}$.

2.4. Through empirical correlations the DMT yields useful information about $K_o$ in soft to medium cohesive deposits. The results in stiff clays and sands look promising but further field and laboratory validation is required.

2.5. The ratio of the excess pore pressure measured during the CPTU to the total net cone resistance ($q_c - \sigma_{vo}$) reflects OCR changes in uniform cohesive deposits.

### 3. DEFORMATION CHARACTERISTICS

3.1. The assessment of the soil moduli in situ, especially in cohesionless deposits is of relevant practical interest. At present it is however not straightforward to connect the obtained moduli to either strain level of the laboratory macroelement or to the strain level for different engineering situations.

3.2. The unload-reload loops performed during the SBP allows reliable values of shear moduli in both clays and sands to be measured. Especially with sands, the "elastic" shear response below yield is of great practical interest because of the well known limitations of laboratory techniques in these soils.

3.3. The DMT allows the constrained modulus M in soft to medium clays and sands to be evaluated. Further developments will involve correlations between dilatometer modulus $E_D$ and G or E'.

3.4. The measured cone resistance $q_c$ in sands appears to be only weakly correlated to their deformability characteristics; thus the reliability of E' vs. $q_c$ relations is rather limited. A review of the calibration chamber and $CK_oD$ triaxial tests performed on predominantly quartz pluvially deposited sands indicates:

(1)

For NC sands: $$1.8 \leq \frac{E'_{25}}{q_c} \leq 2.6$$

(2)

For OC sands: $$6.0 \leq \frac{E'_{25}}{q_c} \leq 19.0$$

### 4. FLOW AND CONSOLIDATION CHARACTERISTICS

4.1. A reliable evaluation of the "mass permeability" in cohesionless deposits may be achieved through a properly programmed and conducted pumping test.

4.2. A large number of in situ techniques are available for assessing flow (k) and consolidation (c) characteristics in cohesive deposits. However when using these parameters in design, the following factors should be considered:

(1)

Disturbance of the surrounding soil caused by insertion of the test device.

(2)

Changes in the effective stress state occurring in the surrounding soil during the test.

(3)

Relation between the specific soil volume being tested and the deposit's macrofabric.

None of the presently available methods allow the k and c to be determined in the range of $\sigma'_v$ well beyond $\sigma'_{vo}$.

4.3. Self-boring devices minimize to a great extent the disturbance of the soil in which k and c are measured.

(1)

At present the self-boring permeameter which allows only outflow tests to be performed yields k and c values representative of the recompression curve.

(2)

Because of possibly nonmonotonic changes of the effective stress around the expanded Camkometer probe, uncertainty exists about the proper applicability of $c_h$ deduced from holding tests.

4.4. Proper analyses of the excess pore pressure and strains or settlements measured in the field offer a chance to reliably assess values of k and c, provided that some preliminary assumptions are properly made. The uncertainties associated with these assumptions may be at least partially overcome by applying appropriate phenomenological fitting techniques to the interpretation of field records.

### 5. DATA ACQUISITION SYSTEMS

5.1. Despite an extremely rapid recent development of hardware, there is still, at least to the writers'knowledge, a need of improvement in the following areas:

(1)

Noise level rejection.

(2)

A/D converters with increasing sensitivity and autoranging.

(3)

Faster relays allowing data collection at rates comparable to computer computation rates.

5.2. Geotechnical engineers should make additional effort to improve data acquisition software by:

(1)
Improving data reduction techniques and increasing their precision.

(2)
Developing "intelligent" data collection schemes by recording only the meaningful data.

(3)
Developing logical retrievable data bases to allow the use of the past measurements to answer future questions.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